EFFECT OF ORIENTATION OF REINFORCEMENT ON STRENGTH OF REINFORCED SOIL

A Thesis Submitted
in Partial Fulfilment of the Requirements
for the Degree of
MASTER OF TECHNOLOGY

by

DINESH KUMAR SINGH

to the

DEPARTMENT OF CIVIL ENGINEERING
INDIAN INSTITUTE OF TECHNOLOGY KANPUR

MAY, 1993

DEDICATED TO

MY PARENTS

10/5/93

CERTIFICATE

It is certified that the work contained in the thesis entitled "EFFECT OF ORIENTATION OF REINFORCEMENT ON STRENGTH OF REINFORCED SOIL" by Dinesh Kumar Singh (Roll No.- 9110308) has been carried out under my supervision and this thesis work has not been submitted elsewhere for the award of a degree.

May 10, 1993

Dr. S. CHANDRA
Associate Professor
Department of Civil Engineering
Indian Institute of Technology
Kanpur, INDIA.

ACKNOWLEDGEMENTS

I express my sincere gratitude to Prof, S. Chandra for suggesting the present thesis work. I am extremely grateful to him for his constant motivation and expert guidance during the work.

I am thankful to Shri Gulab Chand, Shri R.P. Trivedi, Shri A.K. Srivastava and Shri Parashuram for giving all possible cooperation during the experimental work.

I am grateful to my friends Mr. D. K. Pandey, Mr. Taha, Mr. U.C. Johiri, Mr. J.T. Sahu, Mr. D.N. Singh, Mr. S. Srivastava, Mr. Jimmy Thomas, Mr. S. Prasad and other friends for their help during my work.

At last, it gives me immense pleasure to express my deep gratitude to my beloved parents for their constant encouragement.

DINESH KUMAR SINGH

CONTENTS

LIST OF SYMBOLS

C = Cohesion of soil

C_R = Anistropic or psuedo cohesion of reinforced soil

f = Coefficient of friction between soil and reinforcement

K_p = Coefficient of passive earth pressure

K_a = Coefficient of active earth pressure

R = Radius of soil sample

T = Force per unit width of fabric reinforcement

α = Angle of inclination of assumed failure plane

β = Angle of inclination of reinforcement counterclockwise from the major

σ_1 = Major Principal Stress

σ_3 = Minor Principal Stress

σ_R = Stress due to the Reinforcement

ϕ = Angle of Friction of soil

ε = Strain in the Reinforcement of the soil

ε_v = Vertical strain in the soil at peak stress

ν = Poisson's ratio of the soil

LIST OF FIGURES

LIST OF TABLES

ABSTRACT

The effect of orientation of reinforcement on strength-deformation behaviour of soil has been studied by conducting triaxial compression tests on unreinforced and reinforced soil samples with inclined reinforcement under drained condition. Soil and reinforcing material used in the present work are flyash and geotextile respectively.

The experiments have been conducted for four different angles of reinforcement placement and at four different confining stresses.

A simple method has been proposed for predicting the major principal stress at failure by considering the force mobilized in reinforcement. A good correlation has been found between predicted values and test results.

INTRODUCTION

It has been constant endeavour of research workers to put forth innovative ideas to improve mechanical properties of soil to suit the requirement of engineering structure. There are various methods for ground improvement, amongst them, reinforced soil is one of the later developments.

"Reinforced soil" is often treated as a composite material, in which the reinforcement resist tensile stresses and interacts with the soil through friction and/or adhesion. In a broader sense, this term is also used for other mechanical and structural methods of soil improvement such as compression reinforcement and reinforcement by confinement and encapsulation.

In most practical situations the geotextile, as a reinforcing material, are placed in horizontal direction from point of view of convenience of construction. The horizontally placed reinforcement may not be necessarily in the direction of principal tensile strain at all points when applied load is in the vertical direction. Thus the horizontal reinforcement will be inclined to the principal tensile strain direction beyond certain region. Similar situation may arise, more critically, in footing subjected to inclined load resting on horizontally placed reinforced soil.

In triaxial compression testing, principal tensile strain direction is horizontal. In this study, to simulate the condition

of reinforced soil, when reinforcement is inclined to the principal tensile strain direction, drained triaxial compression tests on reinforced soil specimen with reinforcement at varying angles with horizontal have been carried out. Tests have been performed on soil specimen reinforced at four different angles and at four different confining pressures.

Chapter 2 reviews the pertinent literature available on strength deformation behaviour of reinforced soil. Chapter 3 presents the details of experimental work carried out in this work and details of types of material used.

Chapter 4 presents the results obtained from experimental work carried out in this study. The results are analyzed and discussed in chapter 5. The effect of cohesion has been included in the expression for determining the maximum major principal stress. The computed values of maximum major principal stress is found to be in good agreement with the experimental results.

The conclusions drawn from this study have been presented in chapter 6 alongwith the suggestions for future research work.

CHAPTER - 2

LITERATURE REVIEW

2.1 INTRODUCTION:

Soil reinforcement has been used since ancients times. Some of the earlier reinforcement material were natural like tree branches, rope fibres, bamboo strips etc but there were no rational studies to predict the engineering behavior of soil reinforced with natural material. First rational study on reinforced soil was undertaken by Henri Vidal in 1966. Since then several investigators have studied various important aspects related to reinforced soil.

Reinforcing materials are of various types, such as geosynthetics, metallic sheets, nets, strips, synthetic fibers and fibre reinforced plastics. There are various forms of geosynthetics e.g geotextiles, geogrids, geonets, geocomposits. Geotextiles form the largest group of geosynthetics.

2.2 PRINCIPLE OF REINFORCED SOIL:

The essential phenomenon in reinforced soil is the friction between the soil and the reinforcement. Through this friction, the soil transmits the stresses to the reinforcement if the reinforcement is placed in the direction of tension,interaction between the soil and the reinforcement will generate the friction forces. Consequently, tensile stresses will be produced in the reinforcement and corresponding compression will be induced in the soil element as long as there is no slippage between soil and reinforcement. This additional compression will increase the

confining stress, as a result of which soil can take more axial load (Narain, 1985)

The reinforced soil may fail due to either failure of reinforcement or compression failure of soil between reinforcement layers. The failure of reinforcement may be of two types, viz reinforcement tension failure or rupture of reinforcement and reinforcement pullout failure or slippage between soil and the reinforcement.

Al-Omari et al (1988) classified failure of reinforced sand as under-reinforced failure or over-reinforced failure. Under-reinforced failure is that which follows minimum energy option. In this case sand stretches the reinforcement during dilation and eventually the conventional slip plane develops and yields the reinforcement. Over-reinforcement is characterized by rupture of sand-reinforcement bond and the post peak bulging between layers. Over-reinforcement failure may be achieved for same soil and reinforcement at higher confining stress which at lower confining stress gives under-reinforcement failure.

Chandrasekaran et al (1989) observed two types of failure mechanism for the reinforced soil sample under triaxial loading. The first mechanism corresponds to that of a compression failure and second failure mechanism due to rupture of reinforcement.

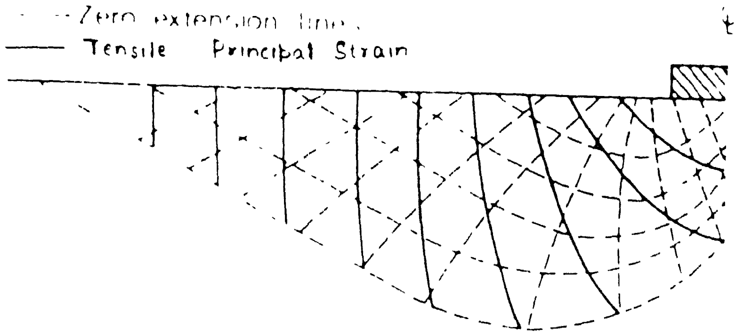
Haussman (1990) states that if reinforced soil fails due to rupture of reinforcement, failure envelop will be parallel to failure envelop of unreinforced soil with increase in cohesion value. If the soil fails by slippage, failure enevelops will not

be parallel and there will be increase in friction angle, which generally occurs at low confining pressures.

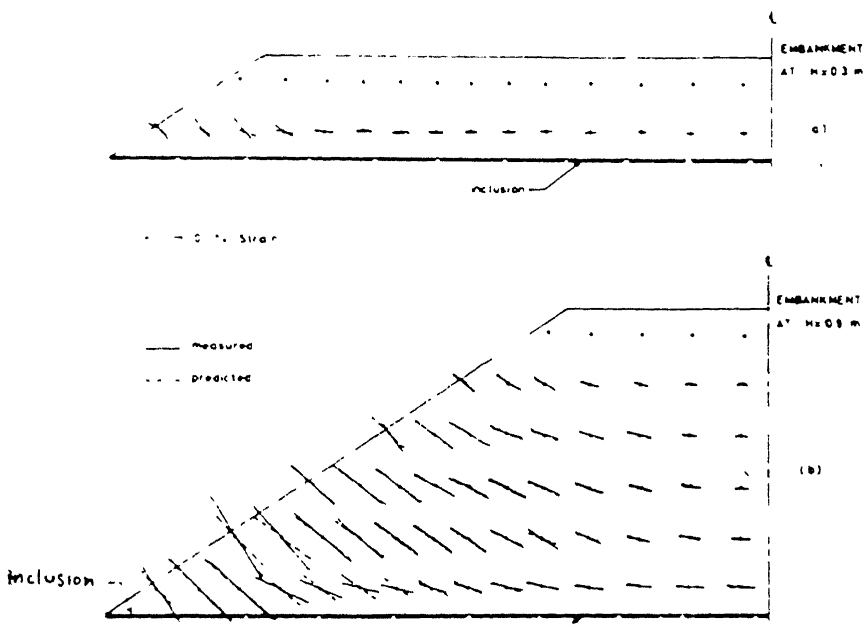
2.3 ORIENTATION OF REINFORCEMENT:

The orientation of reinforcement with respect to the principal tensile strain direction is an important parameter affecting the strength of the reinforced soil. The maximum improvement is achieved by placing reinforcement in direction of principle tensile strain (Mcgown et al 1978, Haussman 1988). The principal tensile strain direction varies from point to point depending upon type of loading and structure (Andrawes et al 1980). The direction of principal tensile strain or direction of minor principal stress for few cases have been shown in fig 2.1 (Andrawes et al, 1980,1983).According to principal tensile strain direction, the ideal pattern of placement of reinforcement should be as indicated in fig 2.2 (Ingold, 1982; Sridharan, 1990). Such a continuous system of reinforcement is not practical in the field.The simplified form of reinforcement pattern is shown in fig 2.3 (Ingold, 1982; Sridharan, 1990).

Practically only metallic reinforcement can be placed in inclined or vertical direction. Placing the geotextile at various inclination, which does not have the required rigidity, is practically difficult. For sake convenience of construction geotextiles are normally placed in horizontal layers. When principal tensile strain direction or principal stress direction varies relative to horizontal, the horizontal reinforcement will not be in direction of principal tensile strain.In this situation horizontally placed reinforcement will be at some angle to the

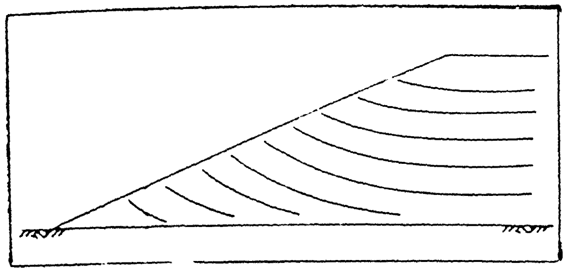


(After Andrawes et al, 1983)

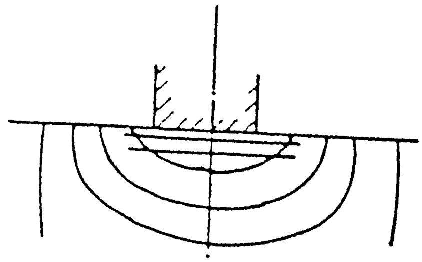


(After Andrews et al. 1980)

Fig.2.1 Orientation of Tensile Principal strain

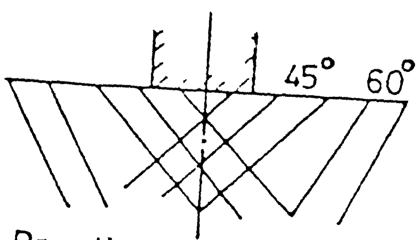


(After Ingold, 1982)

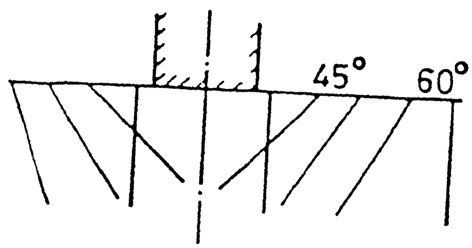


(After Sridharan, 1990)

Fig.2.2 Idealised Reinforcement Orientations



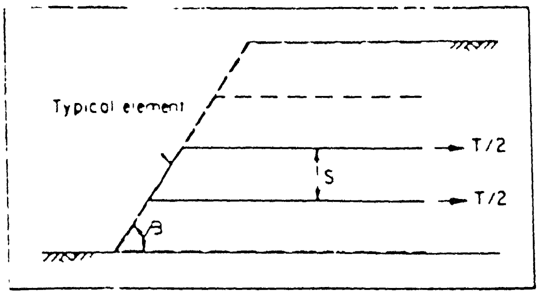
Practical reinforcement pattern (reinforcement placed before structure)



Practical reinforcement pattern (reinforcement placed after structure)

(After Sridharan, 1990)

Fig.2.3a Practical Reinforecment Pattern



(After Ingold, 1982)

Fig.2.3b: Practical reinforcement pattern

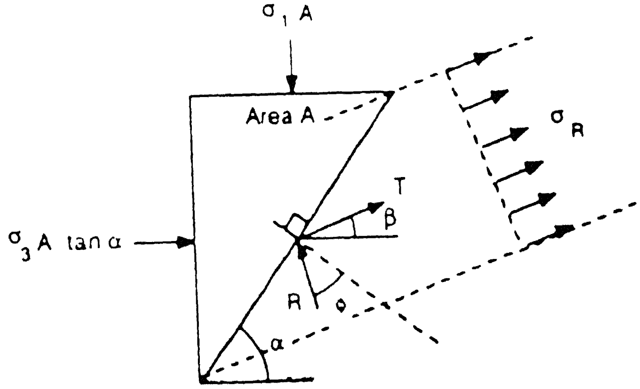


Fig.2.4 Coulomb- type analysis of inclined reinforcement

principal tensile strain direction.

The triaxial compression testing on reinforced soil with horizontal layers of reinforcement simulate the condition of reinforcement in the direction of maximum tensile strain. For simulating the condition of soil reinforced at some angle to the principal tensile direction, triaxial compression tests on soil reinforced with reinforcement placed at angle to horizontal should be performed..

2.4 STRENGTH CHARACTERISTICS OF REINFORCED SOIL:

2.4.1 ANALYTICAL STUDIES:

According to reinforcement direction, reinforcement treated as horizontal or inclined reinforcement. If reinforcement is in plane of major principle plane then it is treated as horizontal otherwise as inclined reinforcement.

Schlosser and Long (1974) proposed the equation for strength of reinforced soil in terms of an anisotropic or pseudo cohesion C_R, which was a function of reinforcement spacing and tensile strength. The strength of reinforced soil is given by

$$\sigma_{1f} = \sigma_3 \, K_p + 2 \, C_R \sqrt{K_p} \qquad (2.1)$$

Gray et al (1982) computed the value of anisotropic or pseudo cohesion from force equilibrium analysis of reinforcement composite.The expression for C_R for horizontal reinforcement is given as:

$$C_R = \frac{\alpha_f \sqrt{K_p}}{2 \, \Delta H} \qquad (2.2)$$

And for inclined reinforcement the expression is given as:

$$\text{Inclined Reinforcement: } C_R = \frac{\alpha_f \, [\, K_p \cos^2\beta - \sin^2\beta \,]}{2\, \Delta H \sqrt{K_p}} \qquad (2.3)$$

Where, α_f = force per unit width of reinforcement at failure(KN/m)

ΔH = spacing between reinforcements in m

β = angle of inclination of reinforcement counterclockwise from the major principal plane in degrees.

Haussman (1976), Haussman and lee (1976) discussed the Mohr-Coulomb interpretation of reinforcement action to cover failure by rupture of reinforcement as well as by reinforcement slippage for both cases of horizontal and inclined reinforcement.

HORIZONTAL REINFORCEMENT: In case of failure by rupture of reinforcement, cohesion intercept C_R is given by ;

$$C_R = \frac{\sigma_R \sqrt{K_p}}{2} \qquad (2.4)$$

In case of slippage between the reinforcement and the soil, there is an increase in friction angle,which is defined by

$$\sin \phi_R = \frac{1 + f - K_a}{1 - f + K_a} \qquad (2.5)$$

INCLINED REINFORCEMENT: Fig 2.4 shows a wedge of soil reinforcement at an angle β to the major principal plane at failure. With an angle α defining the inclination of the assumed failure plane, the equilibrium condition yields;

$$\sigma_1 = \sigma_3 K_1 + \sigma_R K_2 \qquad (2.6)$$

where, $$K_1 = \frac{\tan\alpha}{\tan(\alpha-\phi)} \qquad (2.7)$$

$$K_2 = \frac{\sin(\alpha - \beta)}{\cos\alpha} \left[\frac{\cos\beta}{\tan(\alpha-\phi)} + \sin\beta \right] \qquad (2.8)$$

To find the value of σ_1 at failure, for known value of β, ϕ, σ_3 and σ_R; α is varied so as to find a minimum value of σ_1.

For the case of slippage of the reinforcement, $\sigma_R = f.\sigma_\beta$, where σ_β = normal stress acting on the surface of reinforcement; then σ_1 is given by

$$\sigma_1 = \sigma_3 . \frac{K_1 + f.(\sin^2\beta).K_2}{1 - f.(\cos^2\beta).K_2} \qquad (2.9)$$

Minimizing σ_1 for specific values of ϕ, σ_3, and f by numerical methods indicates that the strength increase can be expressed independently as an increase in the friction angle.

Ingold (1979) made an analytical study on triaxial sample reinforced with horizontal layers of reinforcement. He assumed that the friction mobilised is linear function of the radial distance from the centre of the sample, where the strain is assumed zero. He proposed equations for strength ratio of reinforced soil for different cases of triaxial compression test.

Broms (1988) modified the analytical model of triaxial samples reinforced with horizontal fabrics discs. His assumption is same as that of Ingold(1979), but he proposed to use average value of coefficient of lateral earth pressure in the equation for vertical stress distribution along the reinforcement.

Chandrasekaran et al (1989) used the enhanced confining

pressure concept to study the shear stress distribution along the reinforcement.They proposed an equation for the ultimate strength of an axisymmetrically loaded fabric reinforced sand and another equation for maximum tensile force in the reinforcement which relates to the applied vertical load.

2.4.2 EXPERIMENTAL STUDIES:

2.4.2.1 EFFECT OF HORIZONTAL REINFORCEMENT:

Ingold (1979) conducted triaxial test on reinforced clay. His test results showed that in case of drained test, strength increases with number of layers of reinforcement. In case of rapid shear test at higher aspect ratio (spacing/diameter), the results indicated a strength decrease of about 50%.

Gray et al (1982) investigated experimentally the stress strain behaviour of horizontally reinforced sand in triaxial testing. He observed that above critical confining pressure, failure enevelops for all tests tends to be parallel to each other for the reinforced sand and the friction angle of the soil was not affected by the reinforcement. Although reinforcement with synthetic fibres increases ultimate strength but it tends to reduce the overall stiffness of sand. This tendency was more pronounced as the number of reinforcement layers increased. The reinforcement also tends to increase the amount of strain to reach the peak stress.

Holtz et al (1982) conducted laboratory test to evaluate the creep behaviour of reinforced soil. His short term tests showed that reinforcement increases the ultimate strength, deformation modulus and the angle of internal friction of triaxial sample

composed of dense angular sand.The increase in maximum principal stress difference for reinforced sand decreases with increasing confining pressure and reinforced sand fails at larger axial strain.

Ingold(1983) conducted undrained triaxial testing on reinforced clay. He observed that when fully saturated clay is reinforced with continuous impermeable reinforcing medium and subjected to rapid undrained loading, the reinforcement causes a reduction in strength, rather than an increase. The reduction in strength was also observed in case of permeable reinforcing medium. A limited number of tests on partially saturated clay with impermeable reinforcement again confirmed that strength is reduced in clay with high degree of saturation.As degree of saturation decreases, the strength ratio increases until reaching degree of saturation of approximately 70%. The maximum strength ratio achieved is equal to strength ratio of saturated clay under fully drained condition.

Mandal (1986) conducted conventional triaxial tests on dry sand with horizontal circular strips of woven geotextiles in two layers to determine the stress-strain relationship of woven fabrics. Due to introduction of geotextiles, the angle of internal friction,initial tangent modulus and ultimate strength of triaxial samples improved substantially. The geotextile reinforced sand showed smaller strains at failure than corresponding unreinforced samples.

Mandal (1987) again conducted triaxial tests on cohesive soil reinforced with aluminum discs and mild steel fibres. The studies revealed that the strength increases in cohesive soil samples with

different types and forms of reinforcement. The placement of soil at moisture on dry side of optimum is advantageous for compacted fills of reinforced earth.

Krishnaswamy and Reddy (1988) carried out triaxial compression tests under undrained conditions on reinforced silty clay. He concluded that at very low strains, there is loss in compressive stiffness. The loss in compression stiffness was more pronounced when the number of reinforcement layers were more and the moulding water content was low. The strength ratio decreases with increase in water content.

Srivastava et al(1988) studied the unconfined and triaxial behaviour of geotextile reinforced alluvial silt. They also changed the position of single reinforced layer and found that stress-strain of centrally placed reinforcement dominates the stress-strain behaviour of sample reinforced at 1/4 height of sample.The value of friction angle for different layers vary very little and may be considered almost same in all cases.

Krishnaswamy and Raghavendra (1988) conducted direct shear test on Kaoline silty clay samples reinforced with horizontal layers of reinforcement.Their results indicated that at low water content, a soft geotextile material does not have a significant effect on strength of kaoline but produces loss in strength in silty clay. At higher water contents, strength ratio increases with increasing number of layers and also increases with normal stress. Reinforced samples failed at higher strain.

Wu (1989) reported that the loss of compressive stiffness observed in the triaxial tests were entirely due to compression of

reinforcement.

2.4.2.2 EFFECT OF ORIENTATION OF REINFORCEMENT

McGown et al (1978) studied the effect of inclusion properties and orientation of inclusion on the behaviour of sand using a plain strain unit cell apparatus. They concluded that for maximum improvement inclusion must be placed along the direction of principal tensile strain. The placement of reinforcement possessing only tensile strength in the direction of compressive direct strain does not strengthen the soil. The placement of reinforcement inclined to principal strain direction results in decrease in strength of reinforced soil. The amount of decrease is dependent upon the frictional characteristics of soil inclusion interface and angle of placement of the reinforcement to the direction of principal tensile strain.

Mandal and Divshikar (1988) conducted large scale shear test on dry coarse gravelly sand with inclined reinforcement of galvanized iron sheet. Their test results showed that maximum shear stress occurred at inclination of around $(45^{o} + \phi/2)$ to the horizontal.

Hayashi et al (1988) developed a large versatile shear test apparatus in order to examine the reinforcing mechanism in sand reinforced at different angles of reinforcement.Their test results showed that strength ratio only increases when reinforcing material function as tensile material and when the reinforcing material function as compressive material, the strength of reinforced soil decreases.

Chen & Lee (1990) conducted direct shear test on reinforced soil with changing inclination of reinforcement. From their result, they concluded that inclination for maximum improvement is related to angle of dilation. They also observed that most efficient way of using reinforcement is by placing it in the direction of maximum tensile strain of soil mass.

From these literatures, it can be noted that there is need for predicting the strength deformation behaviour of soil reinforced at an angle to horizontal. In this study experiments on soil reinforced at various angle to horizontal under drained triaxial compression tests have been conducted to study its strength deformation behaviour.

CHAPTER - 3

DETAILS OF EXPERIMENTAL STUDIES

3.1 INTRODUCTION:

In general, soil used in the reinforced earth construction is in compacted condition.The orientation of the reinforcement with respect to the principle stress direction is a significant parameter to studying the behaviour of reinforced soil. In this study, experiments have been conducted to study the strength-deformation behaviour of compacted soil with reinforcement at various angle of inclination to horizontal. The details of the materials used, preparation of sample & types of tests performed are presented in this chapter.

3.2 MATERIALS USED:

3.2.1 SOIL:

Soil used for the present study was fly ash obtained from Panki thermal power station, Kanpur.Table- 3.1 shows some of the characteristics of fly ash.

Table 3.1: Properties of Panki fly ash

Colour	- Grey
Specific Gravity	- 2.17
Plasticity Index	- Non plastic
Particle Size:	
Clay (smaller than 0.002 mm)	= 9 %
Silt (0.002 - 0.075 mm)	= 76 %
Sand (0.075 - 2.0 mm)	= 15 %
Harvard Miniature Compaction Test:	
Optimum Moisture content	= 34 %
Maximum Dry Density	= 1.094 g / cc

3.2.2 REINFORCEMENT:

The reinforcing material used for reinforcing soil is geotextiles manufactured by Gujarat Filament Limited, Banglore. The properties of geotextiles used are given in Table 3.2.

Table 3.2: Properties of the Geotextile

Type of fabric	- Woven permeable polystron
Thickness	- 0.48 mm
Bursting Strength	- 20 kg/cm
Failure strain	- 18 %
Secant Modulus @ 5 % elongation	- 180 kg/cm.

3.3 PREPARATION OF SOIL SPECIMEN :

The remoulded soil specimen were prepared with the help of Harvard Miniature compaction mould. The diameter of mould is 38.1 mm and height is 74.5 mm. For all the tests, the air dried

soil was mixed with the desired quantity of water and wet soil was stored in polythene bags for about 3 days for curing. The optimum moisture content was determined by compacting the soil in compaction mould in four layers of equal height with 25 blows each, pressing the compactor up to 15 kg mark.

The specimen for all triaxial compression tests were made at optimum moisture content and maximum dry density. For unreinforced soil specimen, soil was compacted in similar manner as mentioned above. In reinforced soil specimen, reinforcement were placed at centre of sample in one layer. to place the reinforcement at centre of specimen at various angle, samplers made of aluminum were used. Fig 3.1 shows the shape of sampler. The volume of this sampler is half of the volume of Harvard miniature compaction mould and its inclined plane passes through the centre of sample. First, the sampler was placed inside the compaction mould and soil was compacted in two layers. Then sampler was taken out and the reinforcement was placed inside the mould at proper place and soil was compacted in similar manner. After trimming the sample, it was taken out and tests were conducted.

For determining the friction coefficient between soil and reinforcement, a conventional direct shear test apparatus was used. The size of shear box 6.0 cm square. In upper box, the soil was compacted at optimum moisture content for maximum dry density. At the top of lower box, the geotextiles was placed by gluing it on the perpex sheet and tests were conducted in usual manner.

3.4 DRAINED TRIAXIAL COMPRESSION TEST :

The samples were first consolidated under all round cell pressure.A burette was connected to bottom of sample to measure the volume change. Drainage were permitted from one end and radial boundary.Rate of test was chosen considering the coefficient of consolidation of soil, drainage conditions of sample and failure strains of sample such that during the testing pore pressure did not develop. All tests were conducted on rate of 0.0244 mm/min. Tests were conducted on unreinforced and reinforced soil with reinforcement at varying angle to horizontal. and at confining stress of 2.5, 3.0, 3.5 and 4.5 kg/ cm^2.

3.5 DIRECT SHEAR TEST :

This test was done for determining friction coefficient between geotextile and soil. The shear force was applied at interface between soil and geotextiles. The rate of testing was 0.25 mm/ min. This was done at three normal stress of 1.0, 2.0, 3.0 kg/cm^2.

3.6 TYPES OF TESTS CONDUCTED :

Table-2.3 presents the various types of test conducted.

TABLE 3.3

S.N.	TYPES OF TESTS CONDUCTED	NO. OF TESTS
1.	Particle size distribution	1
2.	Specific Gravity	1
3.	Consolidation	1
4.	Compaction	1
5.	Direct tensile test on reinforcement	1
6.	Direct shear test	3
7.	Triaxial drained test	
	(i) Unreinforced soil	4
	(ii) Soil reinforced at 0°	4
	(iii) Soil reinforced at 20°	4
	(iv) Soil reinforced at 30°	4
	(v) Soil reinforced at 40°	4

Result obtained from these tests has presented in chapter-4.

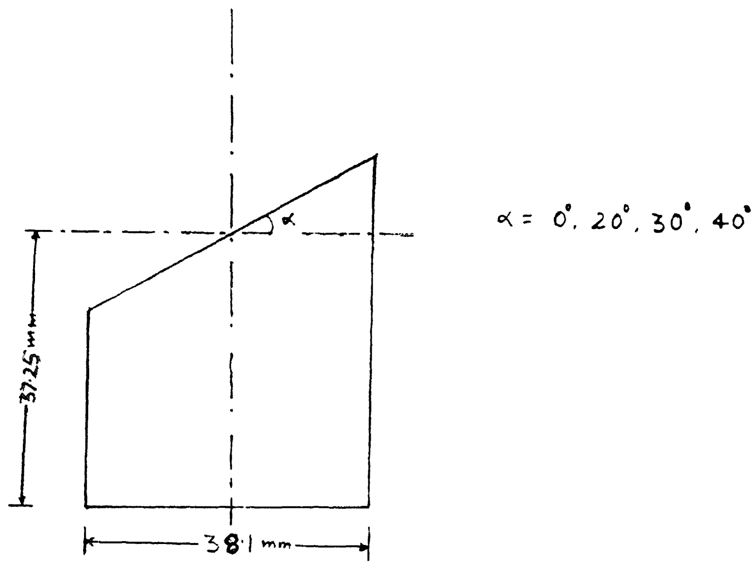


Fig.3.1 Shape of sampler for placing reinforcement soil.

CHAPTER - 4

RESULT OF EXPERIMENTAL STUDIES

4.1 GENERAL:

Results of tests conducted as described in chapter-3 are presented in this chapter. To analyse the strength properties of reinforced soil, drained triaxial tests were conducted. Reinforcement were placed in one layer at centre of sample at various angle to horizontal. Friction coefficient between soil and reinforcement was determined by direct shear test.

4.2 PROPERTIES OF SOIL:

4.2.1 PARTICLE SIZE DISTRIBUTION : The particle size distribution curve of soil used is shown in fig 4.1. Most of the particle of fly ash (76 %) falls in silt fraction. From the curve following interpretation can be drawn.

clay (< 0.002 mm) = 9 %

silt (0.002 - .075 mm) = 76 %

sand (.075 - 4.75mm) = 15 %

Uniformity coefficient Cu =17.5

Coefficient of curvature Cc =5.02

4.2.2 SPECIFIC GRAVITY: The specific gravity of soil was found to be 2.17.

4.2.3 COMPACTION TEST: Dry density vs moisture content curve of compaction test is shown in fig 4.2.

Optimum moisture content = 34.0 %

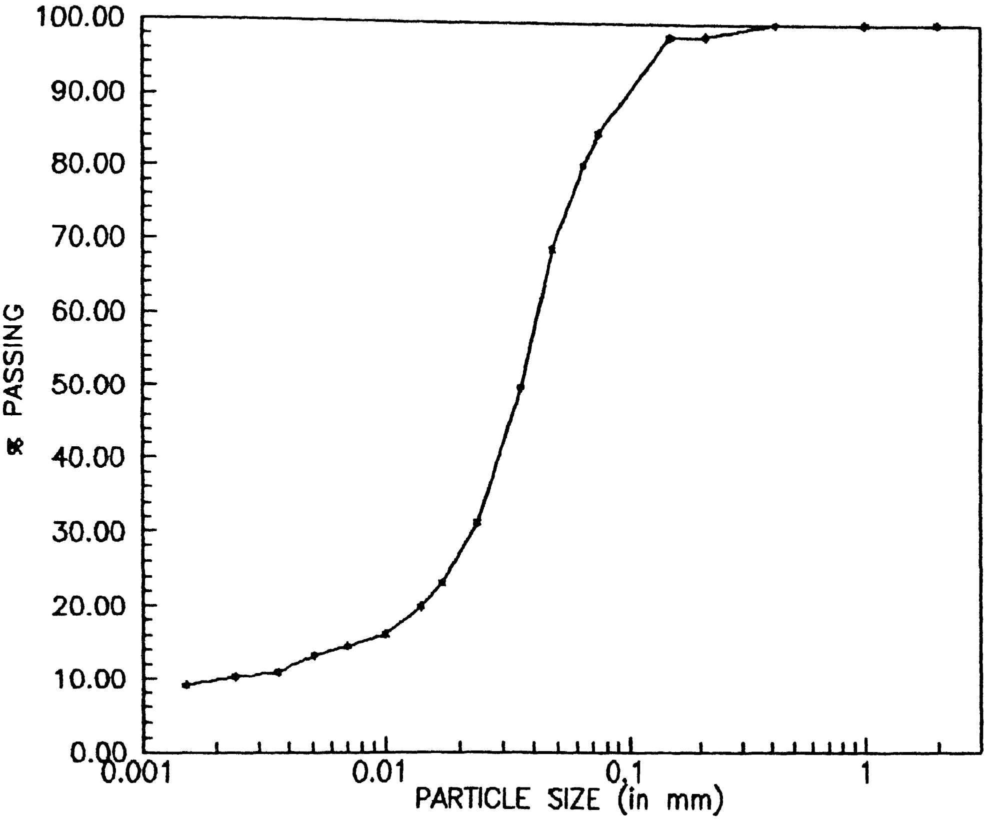


FIG 4.1: PARTICLE SIZE DISTRIBUTION CURVE OF SOIL

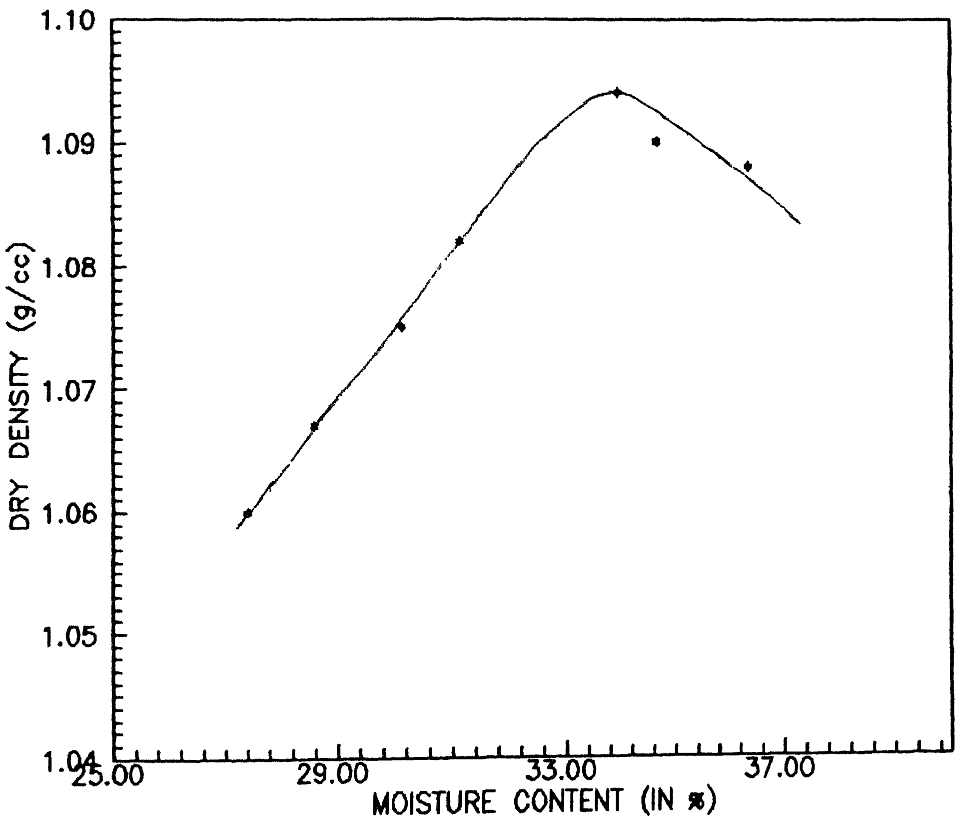


FIG 4.2 CURVE BETWEEN DRY DENSITY AND MOISTURE CONTENT

Maximum dry density = 1.094 gm / cc

4.2.4 COEFFICIENT OF CONSOLIDATION Cv: The coefficient of consolidation at 3.5 kg/cm^2 confining stress was found to be 0.026 cm^2/sec.

4.3 DIRECT TENSILE TEST ON GEOTEXTILE:

Direct tensile test was conducted on geotextile of 25 mm width and 100 mm length with cross head speed of 1.0 mm/min and chart speed of 20 mm/min. Tensile strength was found to be 20 kg/cm at failure strain of 18%. Fig 4.3 shows its force vs strain curve.

4.4 DIRECT SHEAR TEST:

This test was done for determining the coefficient of friction between soil and geotextile. The failure enevelop of this test has been shown in fig 4.4. Table 4.1 presents the results of this test.

TABLE 4.1 RESULT OF DIRECT SHEAR TEST

S.N	NORMAL STRESS (kg/cm^2)	MAXIMUM SHEAR STRESS (kg/cm^2)	COEFFICIENT OF FRICTION
1.	1.0	0.75	
2.	2.0	1.37	.675
3.	3.0	2.10	

4.5 TRIAXIAL DRAINED TEST:

Test results on unreinforced and soil reinforced at angle α = 0^o, 20^o, 30^o and 40^o have been presented in the table 4.2. Deviator Stress vs axial strain curve of various triaxial tests have been shown in fig 4.5 to 4.9.

TABLE 4.2 RESULT OF TRIAXIAL TESTS

S.N.	σ_3 (kg/cm^2)	REINFORCEMENT ANGLE	$(\sigma_1 - \sigma_3)_f$ (kg/cm^2)	FAILURE STRAIN (in %)
1.	2.5	Unreinforced	9.38	7.38
2.	2.5	0^o	11.09	10.07
3.	2.5	20^o	10.68	9.06
4.	2.5	30^o	10.58	8.19
5.	2.5	40^o	8.49	5.37
6.	3.0	Unreinforced	10.94	7.52
7.	3.0	0^o	12.74	8.72
8.	3.0	20^o	11.29	7.85
9.	3.0	30^o	11.92	8.46
10.	3.0	40^o	9.93	5.37
11.	3.5	Unreinforced	11.82	7.72
12.	3.5	0^o	14.71	10.81
13.	3.5	20^o	13.20	9.94
14.	3.5	30^o	12.74	8.39
15.	3.5	40^o	12.03	6.38
16.	4.5	Unreinforced	15.61	9.06
17.	4.5	0^o	18.03	11.07
18.	4.5	20^o	15.61	11.07
19.	4.5	30^o	16.25	10.00
20.	4.5	40^o	15.01	8.37

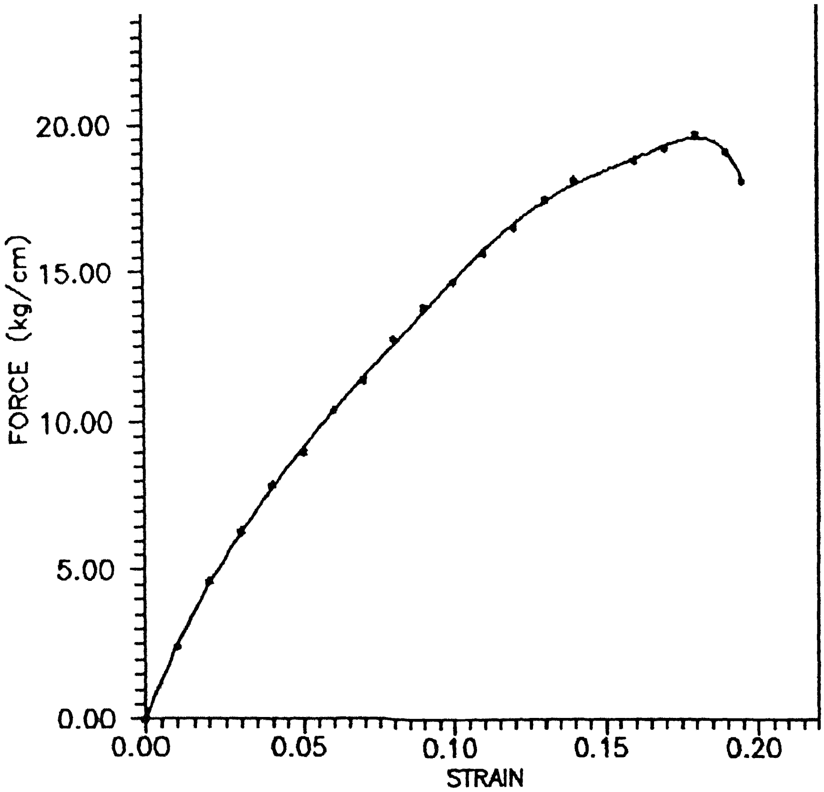


Fig 4.3 Curve of Direct Tensile Test on Geotextile

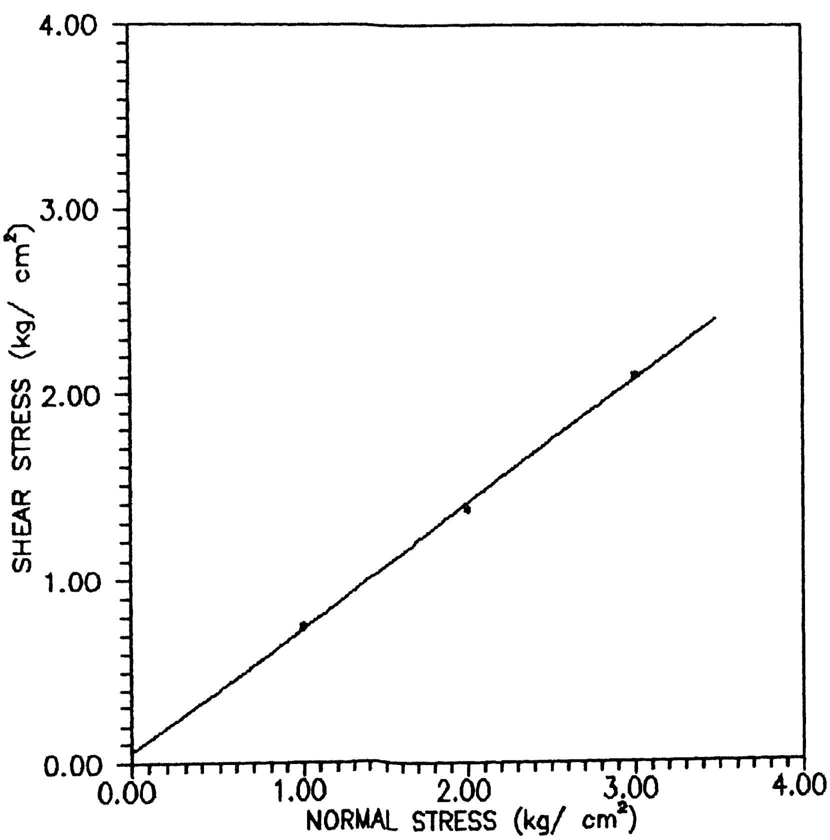


Fig 4.4 Failure Envelop of Direct Shear Test

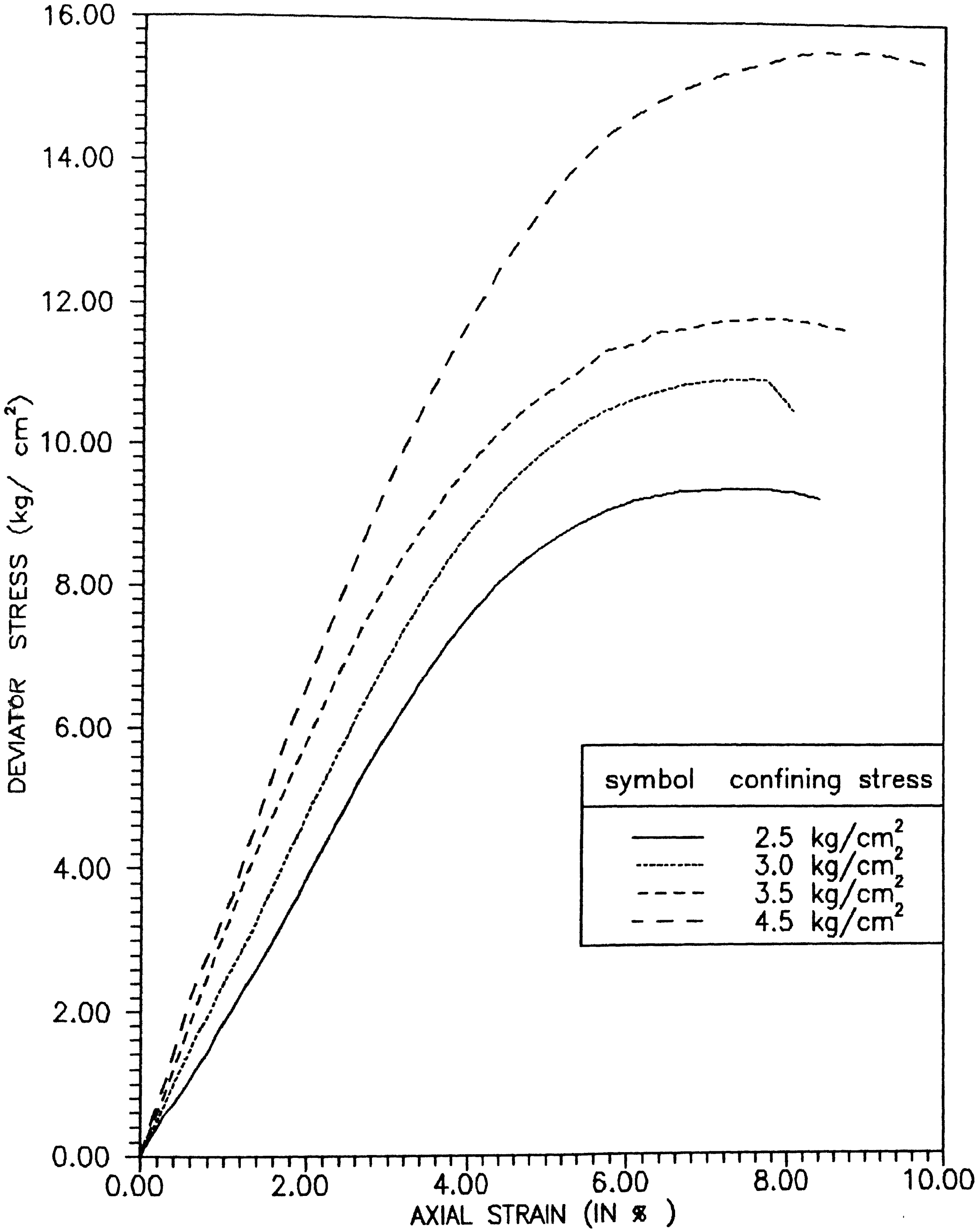


Fig 4.5: Deviator Stress vs Axial Strain
Curve of Unreinforced Soil

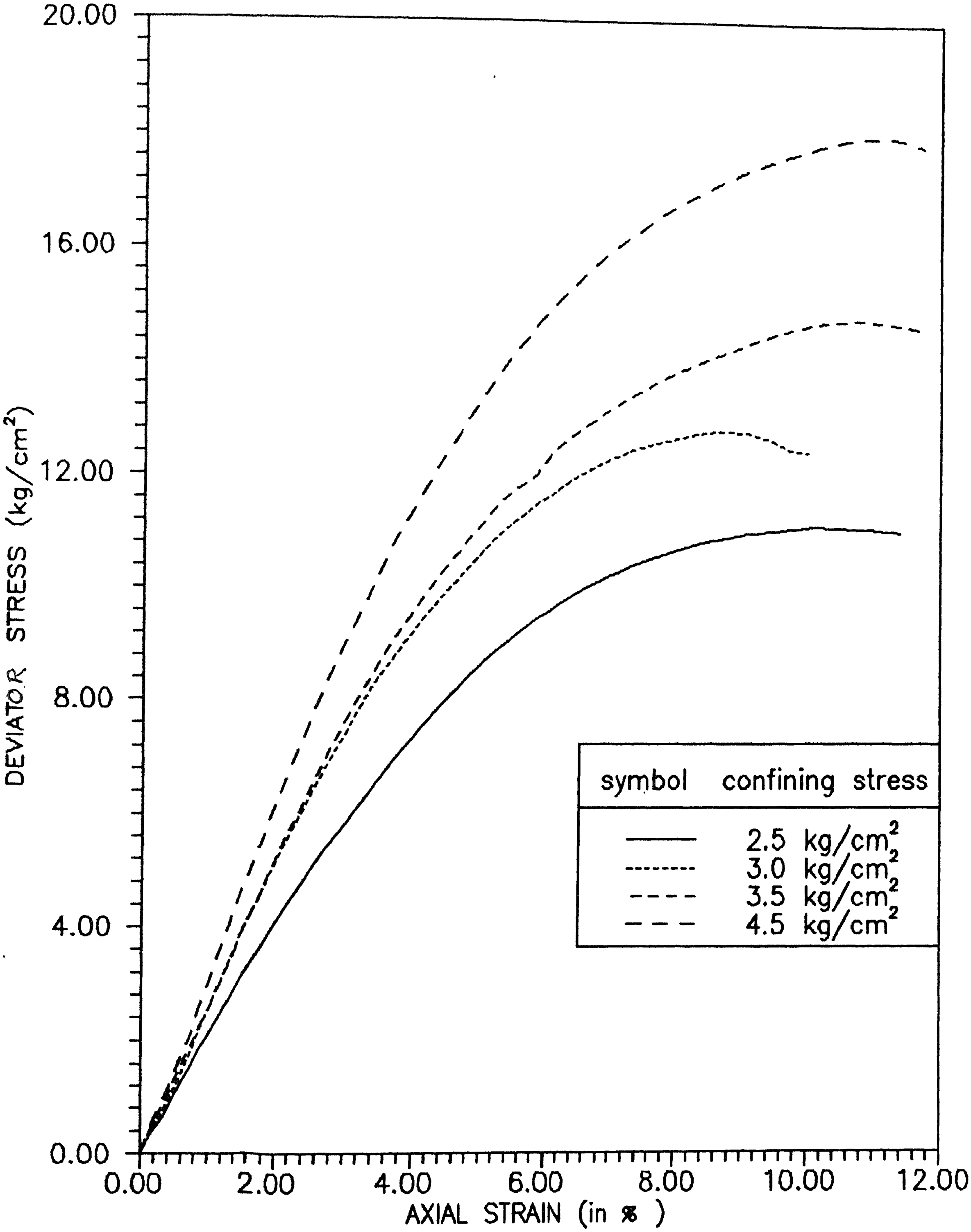


FIG 4.6: Deviator Stress vs axial Strain curve(Reinforcement angle = 0^0)

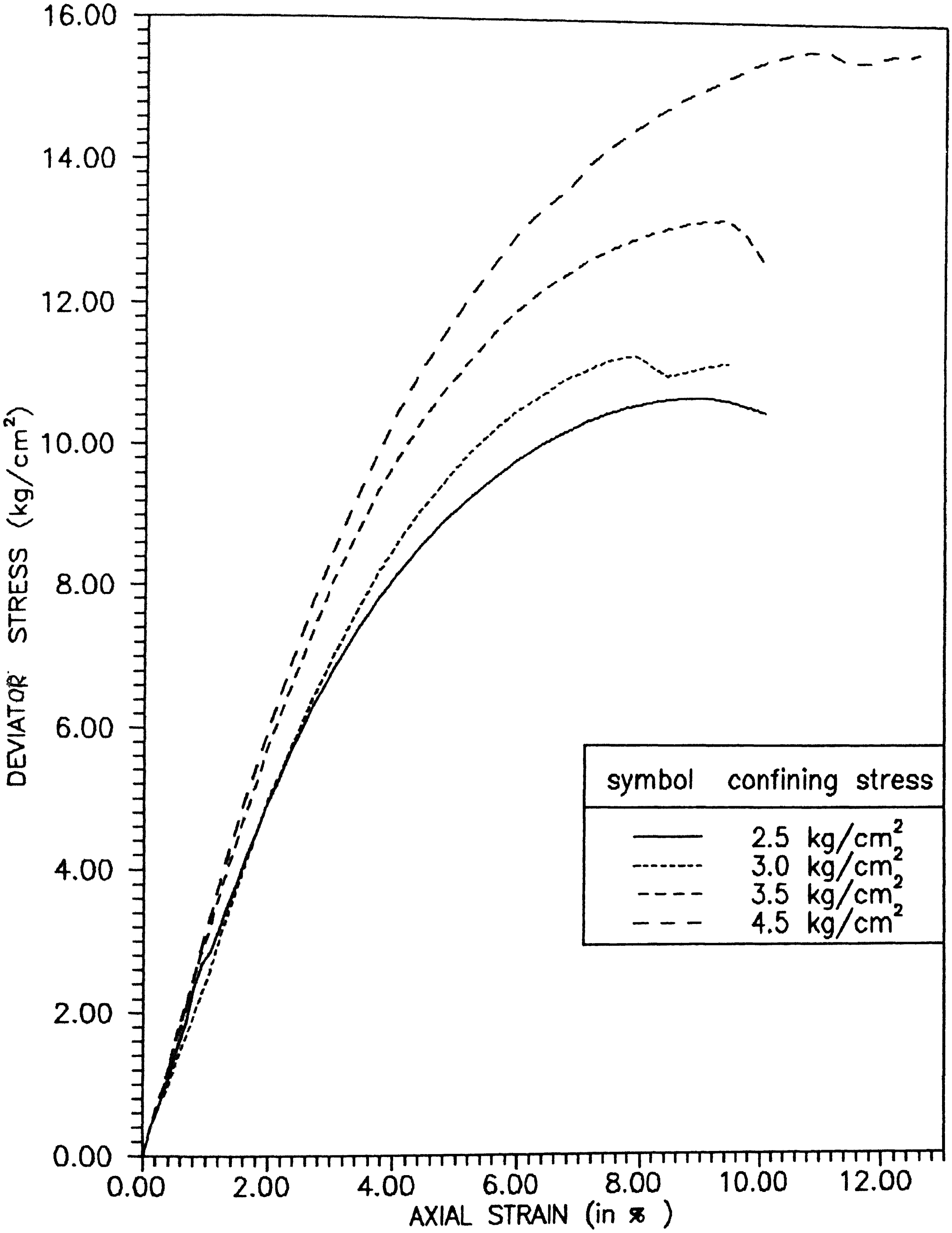


FIG 4.7: Deviator Stress vs axial Strain curve (for reinforcement angle $=20^0$)

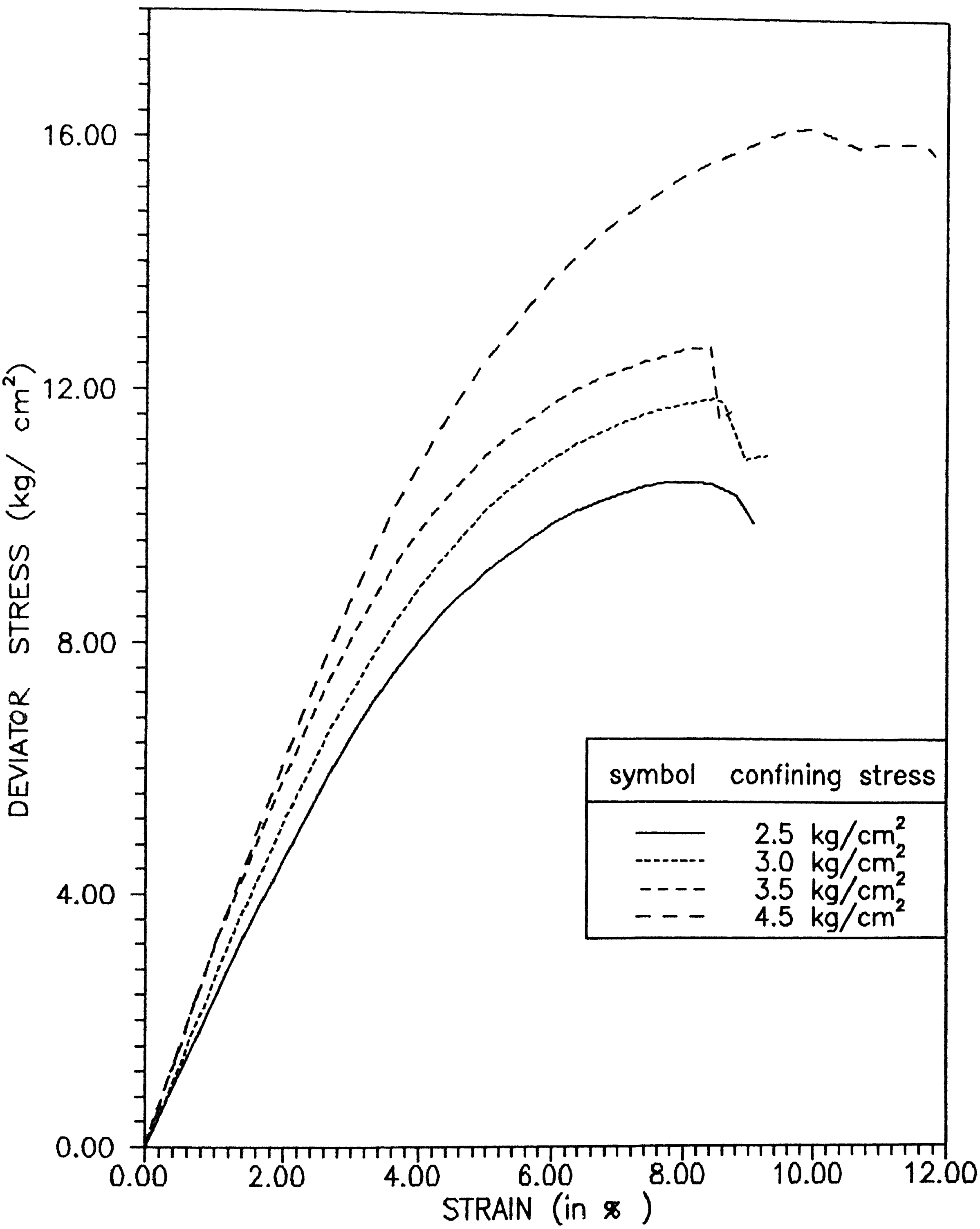


Fig 4.8: Deviator Stress vs Axial Strain Curve
(For Reinforcement Angle $=30^0$)

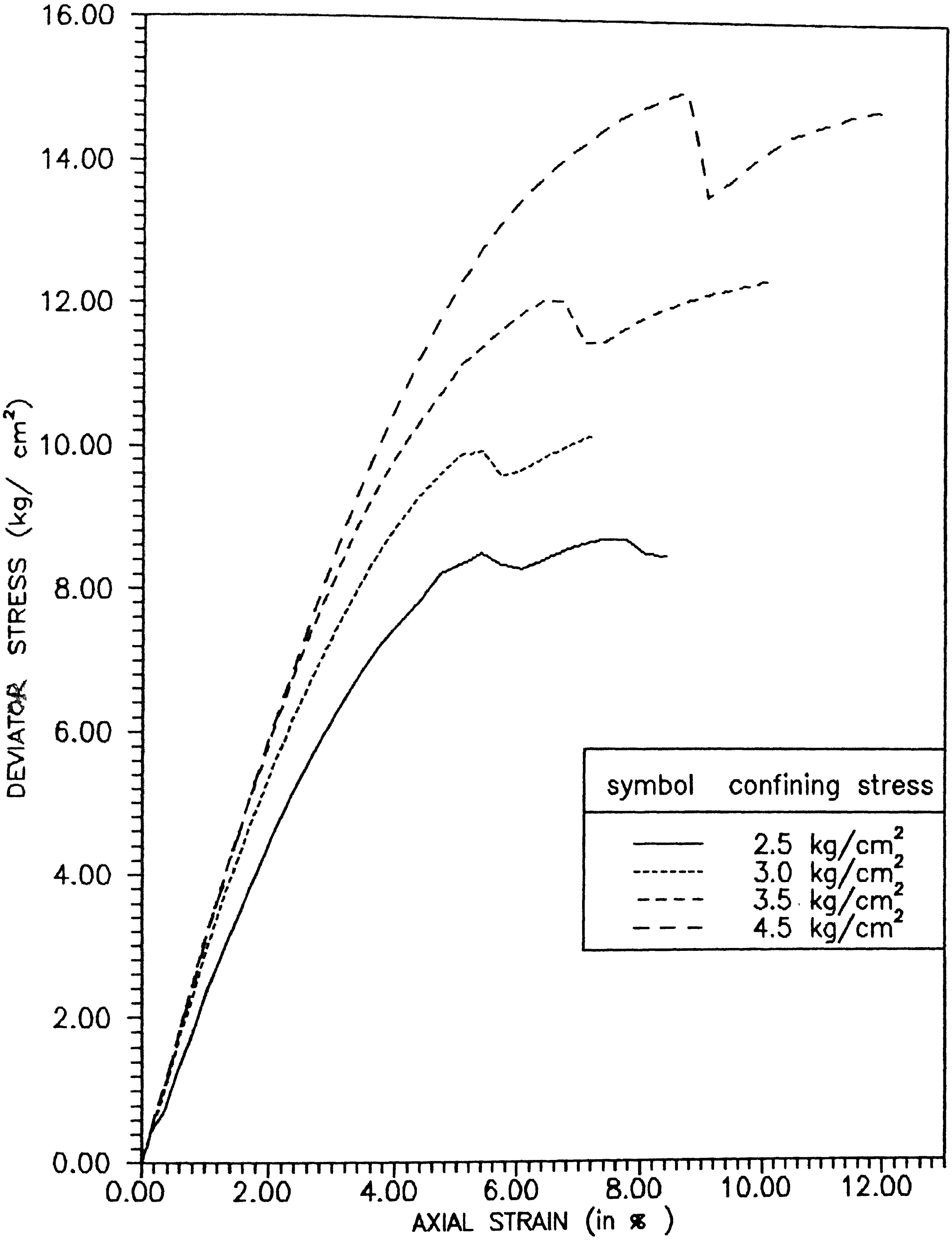


Fig 4.9: Deviator Stress vs Axial Strain Curve
(For Reinforcement Angle $=40^0$)

CHAPTER - 5

ANALYSIS AND DISCUSSION OF TEST RESULTS

5.1 General:

Test results presented in Chapter 4 are analysed in this Chapter. First of all an analytical model has been proposed. Using this model predictions are made and then analytical results and experimental results have been compared. Lastly, strength and deformation behaviour has been discussed in this chapter.

5.2 Analysis:

As earlier discussed in literature review, horizontally placed reinforcement may be either in the direction of major principal plane or at some angle to it, which may be treated as inclined reinforcement. This is due to the fact that, in many situations principal stress direction varies relative to the placement of reinforcement. And this can be analysed using Mohr-Coulomb analysis.

Fig. 5.1 shows the wedge of soil reinforced at an angle β to the major principal plane. Failure plane is assumed at an inclination of α from horizontal. σ_R is the stress developed in reinforcement at failure and σ_1 & σ_3 are major and minor principal stress acting on the wedge at failure and τ and σ are shear stress and normal stress acting on the surface of wedge.

Considering force equilibrium in y direction,

$$\Sigma F_y = 0$$

$$\begin{aligned}\tau.A.\sec\alpha = {} & \sigma_1.A.\sin\alpha - \sigma_3.A.\tan\alpha.\cos\alpha \\ & - \sigma_R.A.\sin(\alpha-\beta).\cos\beta.\cos\alpha/\cos\alpha \\ & - \sigma_R.A.\sin(\alpha-\beta).\sin\beta.\sin\alpha/\cos\alpha \qquad (5.1)\end{aligned}$$

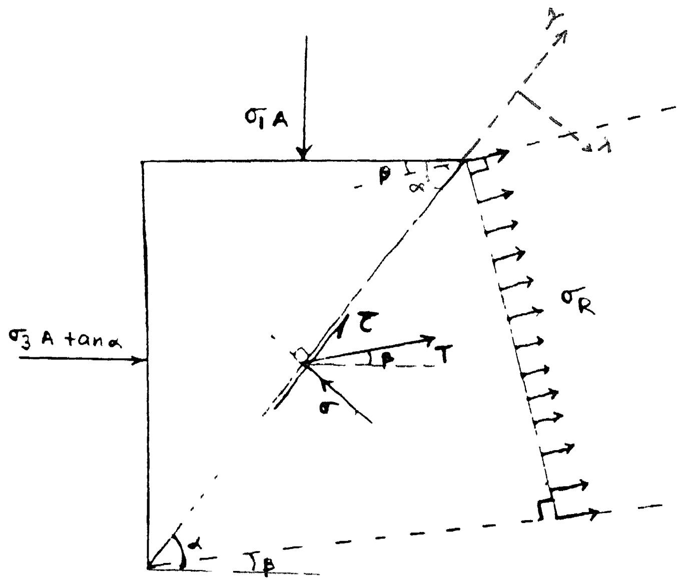


Fig 5.1 Wedge of Soil Reinforced at angle β

Simplifying equation (5.1), we get,

$$\tau = [\{\sigma_1 - \sigma_R.\sin(\alpha-\beta).\sin\beta/\cos\alpha\} - \{\sigma_3+\sigma_R.\sin(\alpha-\beta).\cos\beta./\sin\alpha\}]\sin 2\alpha / 2 \quad (5.2)$$

Similarly summing up the forces in x-direction results,

$$\Sigma F_x = 0$$

$$\sigma.A.\sec\alpha = \sigma_1.A.\cos\alpha + \sigma_3.A.\tan\alpha.\sin\alpha + \sigma_R.A.\sin(\alpha-\beta).\cos\beta.\sin\alpha/\cos\alpha - \sigma_R.A.\sin(\alpha-\beta).\sin\beta.\cos\alpha/\cos\alpha \quad (5.3)$$

Simplification of (5.3) leads to,

$$\sigma = [\sigma_1 - \sigma_R.\sin(\alpha-\beta).\sin\beta/\cos\alpha]\cos^2\alpha + [\sigma_3 + \sigma_R.\sin(\alpha-\beta).\cos\beta/\sin\alpha]\sin^2\alpha \quad (5.4)$$

Taking,

$$F_1 = [\sigma_1 - \sigma_R.\sin(\alpha-\beta).\sin\beta/\cos\alpha] \quad (5.5a)$$

$$F_2 = [\sigma_3 + \sigma_R.\sin(\alpha-\beta).\cos\beta/\sin\alpha] \quad (5.5b)$$

Substituting these values of F_1 and F_2 in equations (5.2) & (5.4)

$$\tau = [F_1 - F_2].\sin 2\alpha / 2 \quad (5.6)$$

$$\sigma = F_1 \cos^2\alpha + F_2 \sin^2\alpha \quad (5.7)$$

Assuming, Mohr-Coulomb failure criteria, the normal and shear stresses are related as,

$$\tau = c + \sigma.\tan\phi \quad (5.8)$$

Substituting values of τ and σ from equations (5.6) and (5.7) in (5.8) and on rearranging,

or $F_1 = c.\cos\phi/[\cos\alpha.\sin(\alpha-\phi)] + F_2.[\sin\alpha.\cos(\alpha-\phi)]/[\cos\alpha.\sin(\alpha-\phi)]$

or $F_1 = c.\cos\phi/[\cos\alpha.\sin(\alpha-\phi)] + F_2.\tan\alpha/\tan(\alpha-\phi)$ (5.9)

Substituting the values of F_1 and F_2 from equations (5.5a) and (5.5b) in equation (5.9), it can be simplified as,

$$\sigma_1 = \sigma_3 . k_1 + \sigma_R . k_2 + c . k_3 \qquad (5.10)$$

where $k_1 = \tan\alpha / \tan(\alpha - \phi)$ (5.11)

$$k_2 = \sin(\alpha - \beta) . [\cos\beta / \tan(\alpha - \phi) + \sin\beta] / \cos\alpha \qquad (5.12)$$

$$k_3 = \cos\phi / [\cos\alpha . \sin(\alpha - \phi)] \qquad (5.13)$$

Constants k_1 and k_2 are same as in equation 2.6 and k_3 appears in this equation due to consideration of cohesion in the analysis.

In order to locate actual failure plane, value of α has been varied within known values of σ_3 and σ_R and the value of α which corresponds to the minimum value of σ_1 has been adopted. This minimum value σ_1 will be ultimate strength of soil.

Haussman and Lee (1976) assumed that at failure,reinforcement ruptures rather than stretching. The chance of breaking, however in most of the situations of a reinforcement fabric with low modulus values are very less. The polystron geotextile, used in this study, breaks at a strain of 18 %. In the triaxial tests conducted on reinforced fly ash samples, the vertical strains observed at peak stresses are only of the order of 10 %, which is much smaller than the failure strain of the reinforcement. The actual strains developed in reinforcement will be even lower than it; depending on the poisson's ratio of soil and the angle of placing of reinforcement. The actual strains developed in reinforcement govern the amount of mobilised tensile resistance. The tensile resistance in the reinforcement can be computed as

follows:

$$T = (J_{sec})_{\varepsilon} . \varepsilon \qquad (5.14)$$

and
$$\varepsilon = \varepsilon_v (\nu . \cos^2 \beta - \sin^2 \beta) \qquad (5.15)$$

where T = Force per unit width of fabric reinforcement corresponding to strain in reinforcement, kg/cm

ε = strain in reinforcement

ε_v = vertical strain in the soil at peak stress

ν = Poisson's ratio of soil.

$(J_{sec})_{\varepsilon}$ = secant modulus of fabric reinforcement between elongation 0 to ε in kg/cm

Above relation assumes that there is no slip at the fabric -soil interface.

and
$$\sigma_R = \frac{T . \cos\alpha}{\sin(\alpha - \beta) . R} \qquad (5.16)$$

where R = radius of sample in cm.

5.3 COMPARISON BETWEEN TEST RESULTS AND PREDICTED RESULT:

Comparison between maximum σ_1 observed and σ_1 predicted by equation (10) to (16) is compared in fig 5.2. The mobilised tensile strength in the reinforcement was computed according to equation (14) and (15) using stress-strain relation for fabric from fig 4.4.and value of poisson's ratio is assumed to be 0.35. It is observed that mobilised tensile strength of fabric at peak stress in the fly ash is only a small fraction (< 25%) of its maximum strength. Similar observation were made by Gray et al (1982).

5.4 SHEAR STRENGTH PARAMETERS OF REINFORCED SOIL:

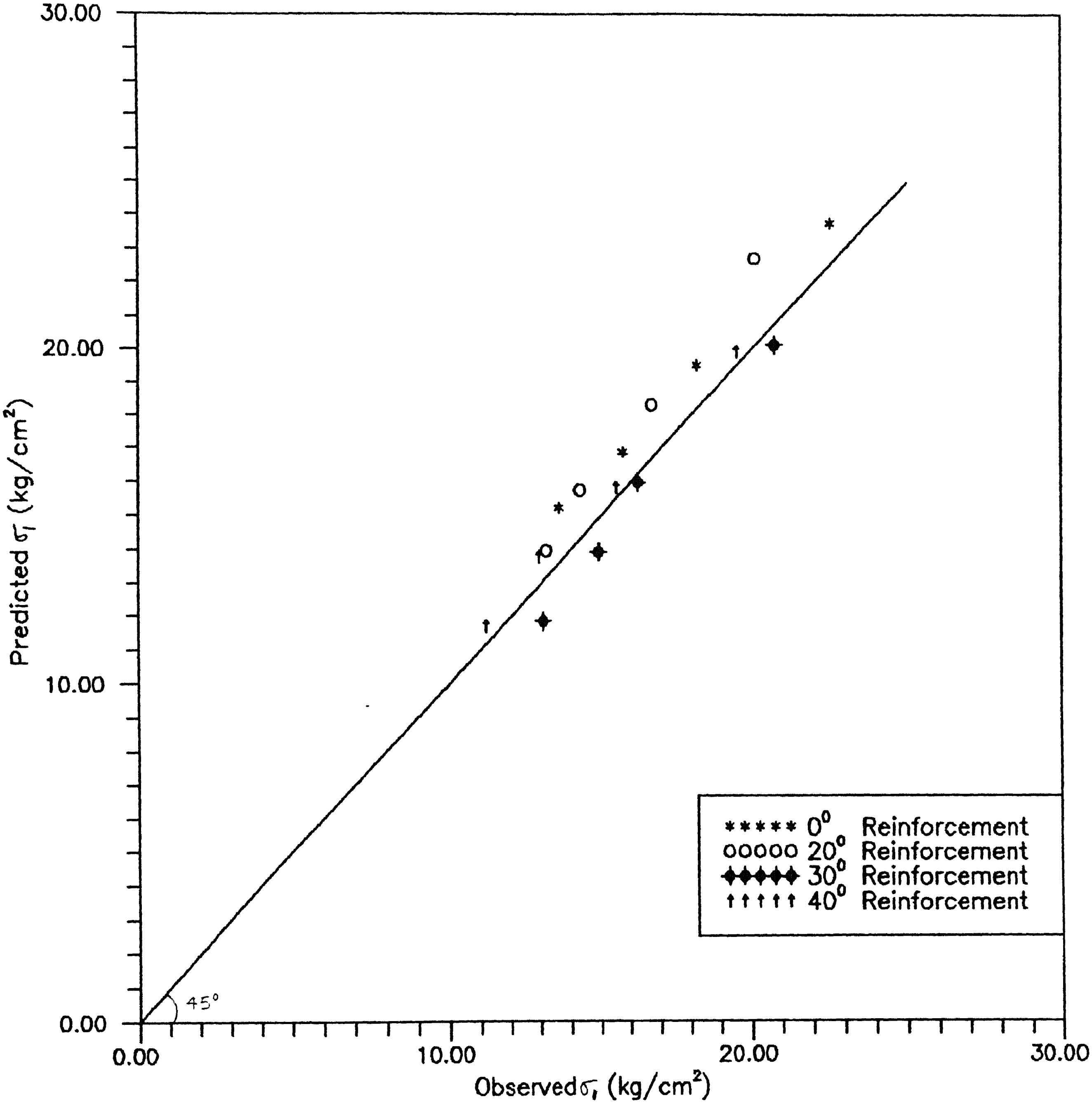


Fig 5.2 Comparison between Predicted σ_1 and Obsrved σ_1

The failure enevelops for each case of unreinforced soil, and soil reinforced at various angles have been plotted in fig 5.3 t 5.6. The results are presented in table 5.1.

Table 5.1: Shear strength parameters of reinforced soil

S.N.	Type of test	Range of σ_3	c in kg/cm^2	ϕ
1.	unreinforced		0.354	37.5^o
2.	0^o reinforcement	2.5 to	0.702	38.4^o
3.	20^o reinforcement	4.5 kg/cm^2	0.550	36.9^o
4.	30^o reinforcement		0.371	38.5^o
5.	40^o reinforcement		0.215	38.1

From the table 5.1, it can be observed that, there is very lit le variance in value of ϕ. Therefore, it can be interpreted that, failure envelop of reinforced soil in the stress range of $2.5 \le \sigma_3 \le 4.5$ is almost parallel.

It has been also found that when reinforcement is placed horizontally, there is maximum increase in value of c, and it decreases as reinforcement angle increases. At $\alpha = 40^o$, value of c of reinforced soil attains smallest value and is less than that of unreinforced soil. It is so because, at 40^o angle of inclination, reinforcement is in compression and geotextile, which only posses tensile strength, is not contributing to strength of reinforced soil.

5.5 Strength Ratio:

Strength ratio of reinforced soil is defined as:

$$\text{Strength Ratio} = \frac{(\sigma_1 - \sigma_3)_{reinforced}}{(\sigma_1 - \sigma_3)_{unreinforced}}$$

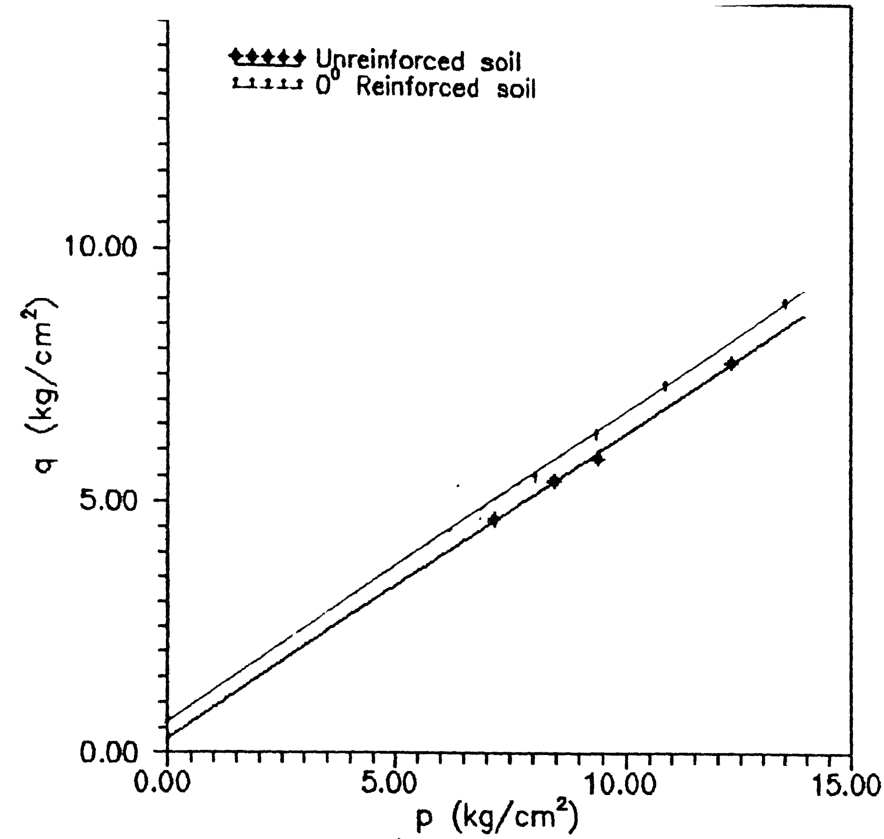


Fig 5.3 : Failure Envelops of Unreinforced and 0^0 Reinforced Soil

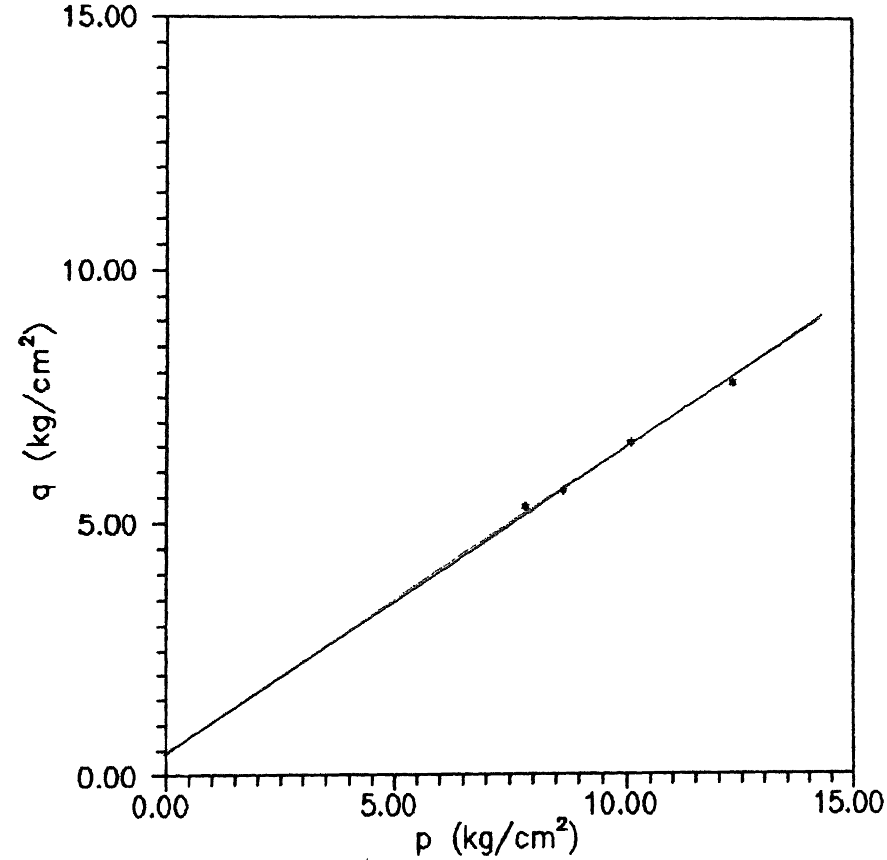


Fig 5.4: Failure Envelop of Reinforced Soil (Angle of Reinforcement= 20^o)

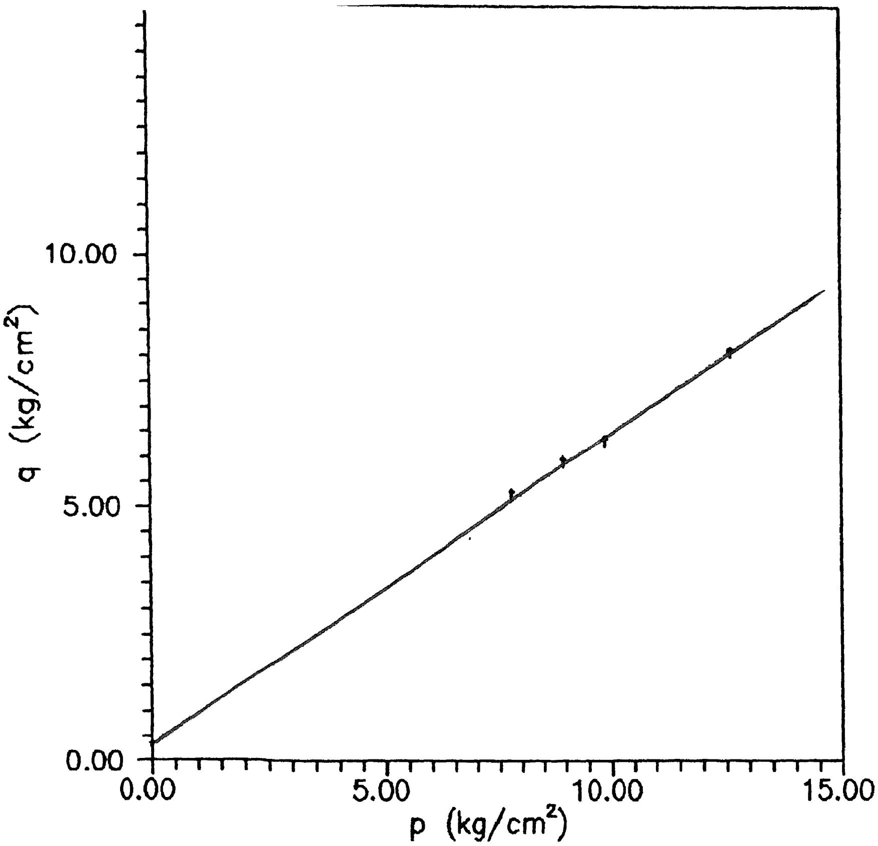


Fig 5.5: Failure Envelop of Reinforced Soil
(Angle of Reinforcement= 30°)

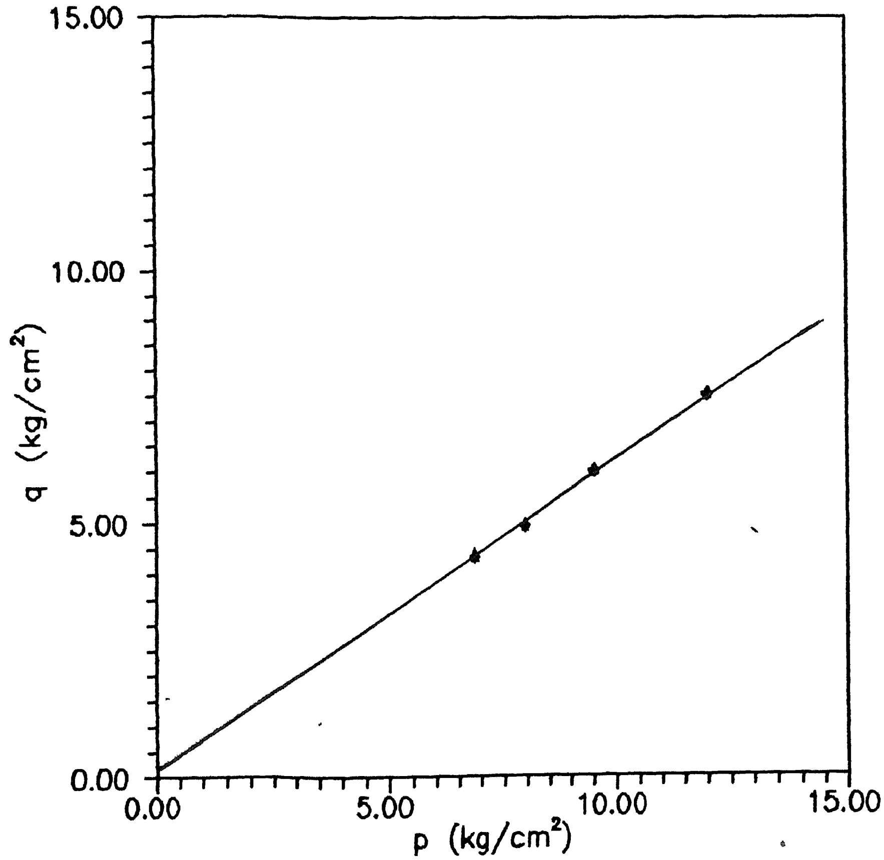


Fig 5.6: Failure Envelop of Reinforced Soil
(Angle of Reinforcement= 40°)

Strength ratio of reinforced soil obtained from various tests have been presented in Table 5.2.

Table 5.2: Strength ratio of reinforced soil

S.No.	Angle of reinforcement	Strength ratio for different σ_a (kg/cm^2)			
		2.5	3.0	3.5	4.5
1	0^o	1.18	1.16	1.24	1.16
2	20^o	1.14	1.03	1.12	1.00
3	30^o	1.13	1.09	1.08	1.04
4	40^o	0.91	0.91	1.02	0.96

From Table 5.2, it can be interpreted that strength ratio is maximum at 0^o and it decreases with increase in angle of inclination of the reinforcement. Strength ratio of the soil reinforced at 20^o and 30^o have very little difference. At 40^o angle of inclination of the reinforcement, strength ratio is less than one, implying that at 40^o angle of inclination of the reinforcement, the strength gets decreased to a value smaller than that of unreinforced soil.

From both Tables 5.1 & 5.2, it can be observed that the maximum improvement due to reinforcement will be achieved when it is placed in the direction of principal tensile strain, which is horizontal in case of triaxial compression tests. Placing reinforcement away from it, strength of reinforcement will decrease and when reinforcement is placed in the plane of compressive strain, strength of reinforced soil will be less than that of unreinforced soil.

5.6 Modes of Failure:

In present study, it is found that reinforced soil specimen fail by compression failure mechanism, as described by Chandrasekaran et al. (1989). Since, mobilised tensile force developed in reinforcement is lower than the breaking strength of the reinforcement, there is no chance of failure by rupture of reinforcement. During the test, it is also observed that there is formation of slip plane at failure. The ultimate strength of reinforced soil, which failed by compression failure of soil, is governed by tensile force mobilized in the reinforcement.

The failure strain of horizontally reinforced soil is found more than unreinforced soil. With increase in angle to horizontal, failure strain decreases and at 40^{o} inclination of reinforcement, failure strain is less than that of the unreinforced soil.

5.7 Deformation Behaviour of reinforced soil:

Fig 5.7 to 5.10 shows the stress-strain response of fly ash at different angle of inclination of the reinforcement and at different confining stress level. From these Figs. It can be observed that there is increase in compressive stiffness of the soil due to reinforcement at 2.5 and 3.0 kg/cm^2 confining stress but at 3.5 and 4.5 kg/cm^2 confining stress there is decrease in compression stiffness of soil due to reinforcement.

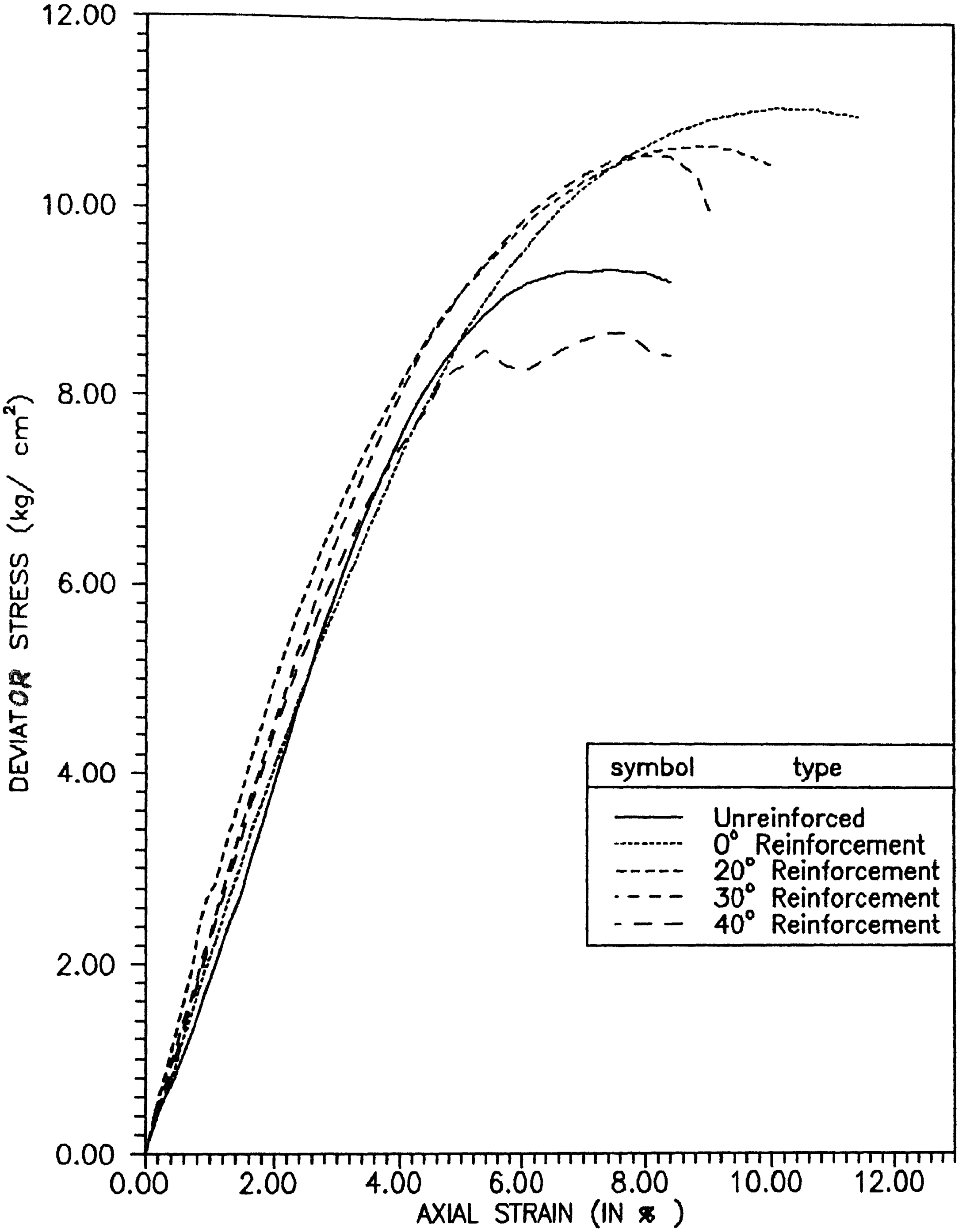


Fig 5.7: Deviator Stress vs Axial Strain Curve of Soil (For Confining stress = 2.5 kg/cm^2)

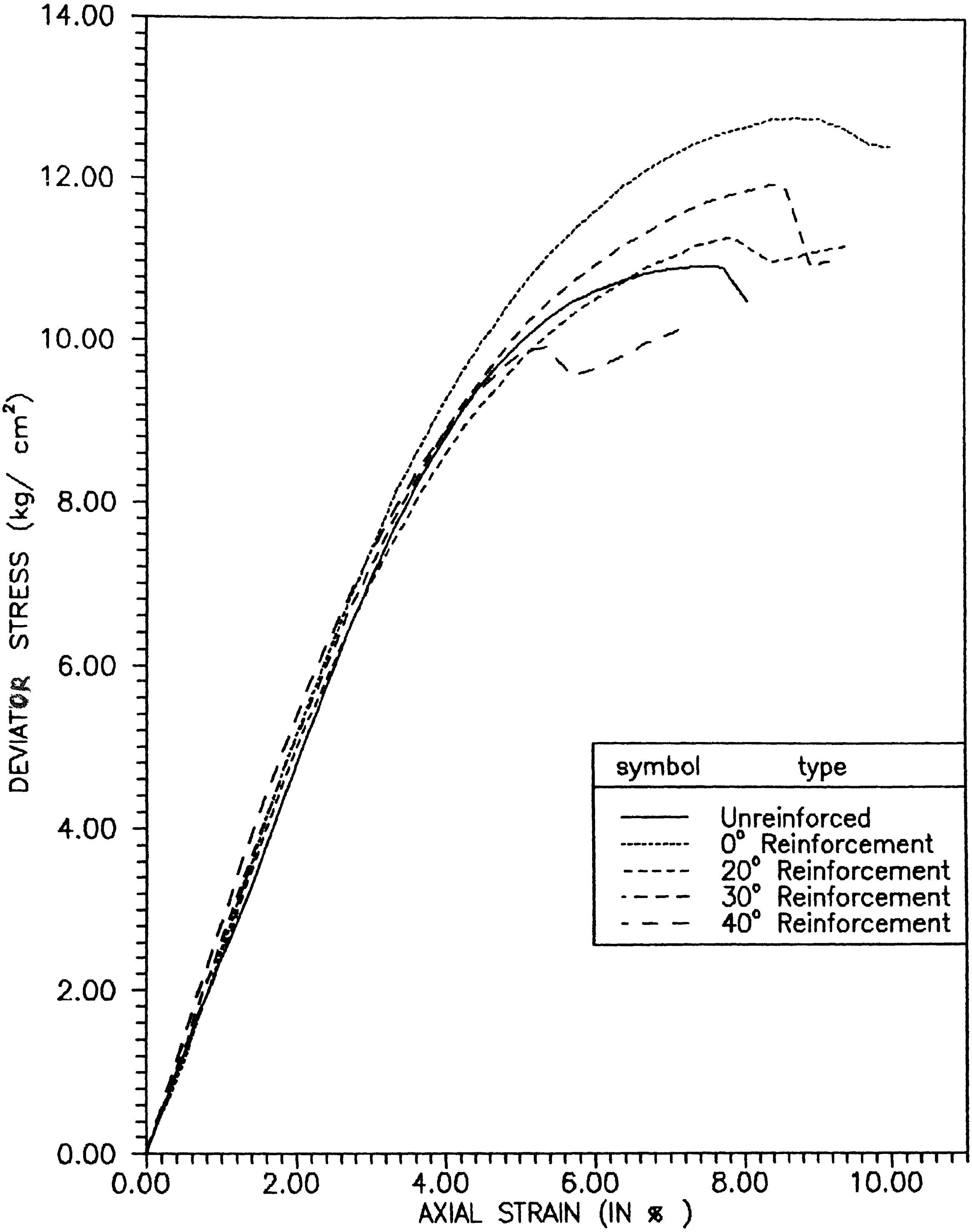


Fig 5.8: Deviator Stress vs Axial Strain Curve of Soil
(For Confining stress = 3.0 kg/cm²)

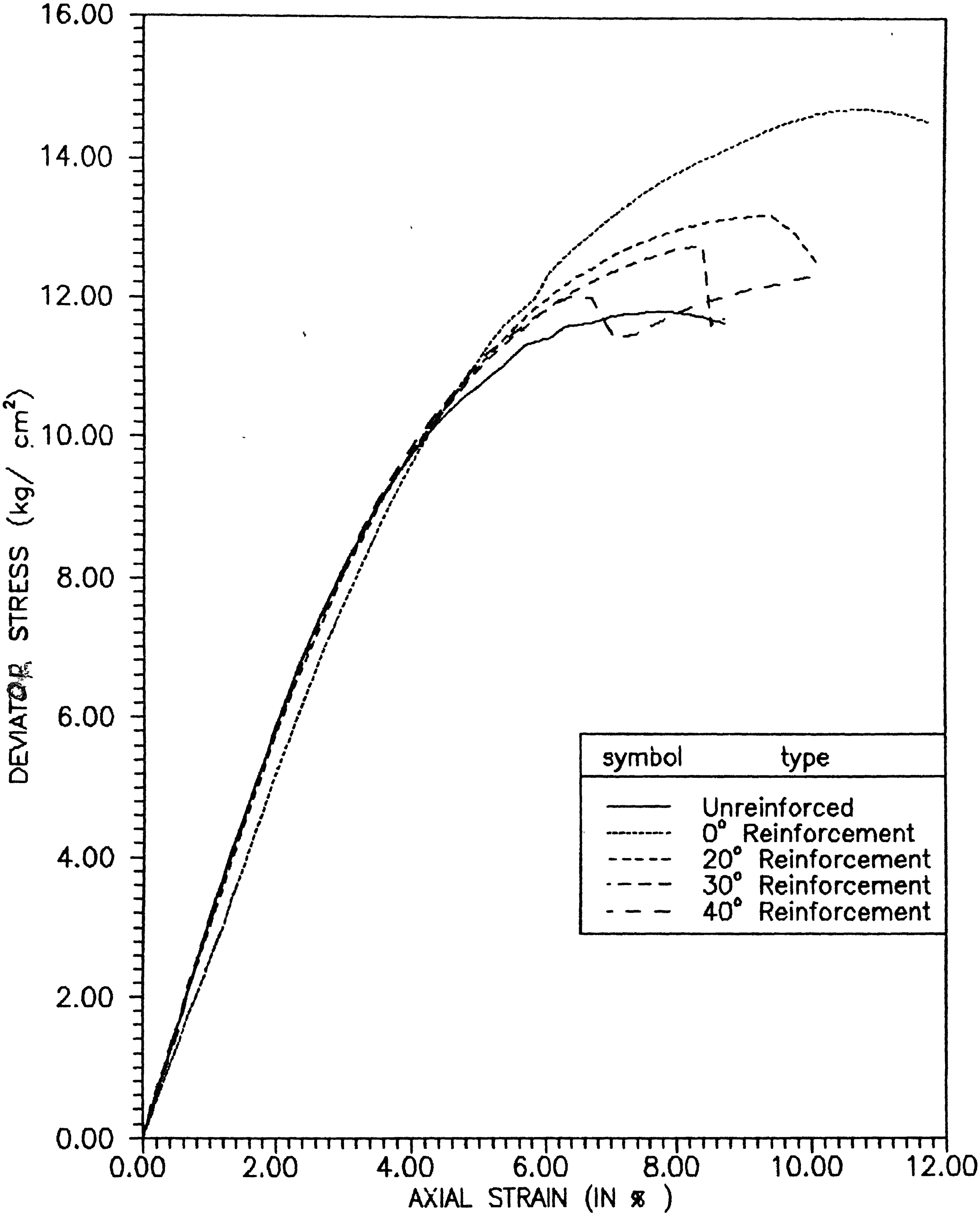


Fig 5.9 : Deviator Stress vs Axial Strain Curve of Soil
(For Confining stress = 3.5 kg/cm^2)

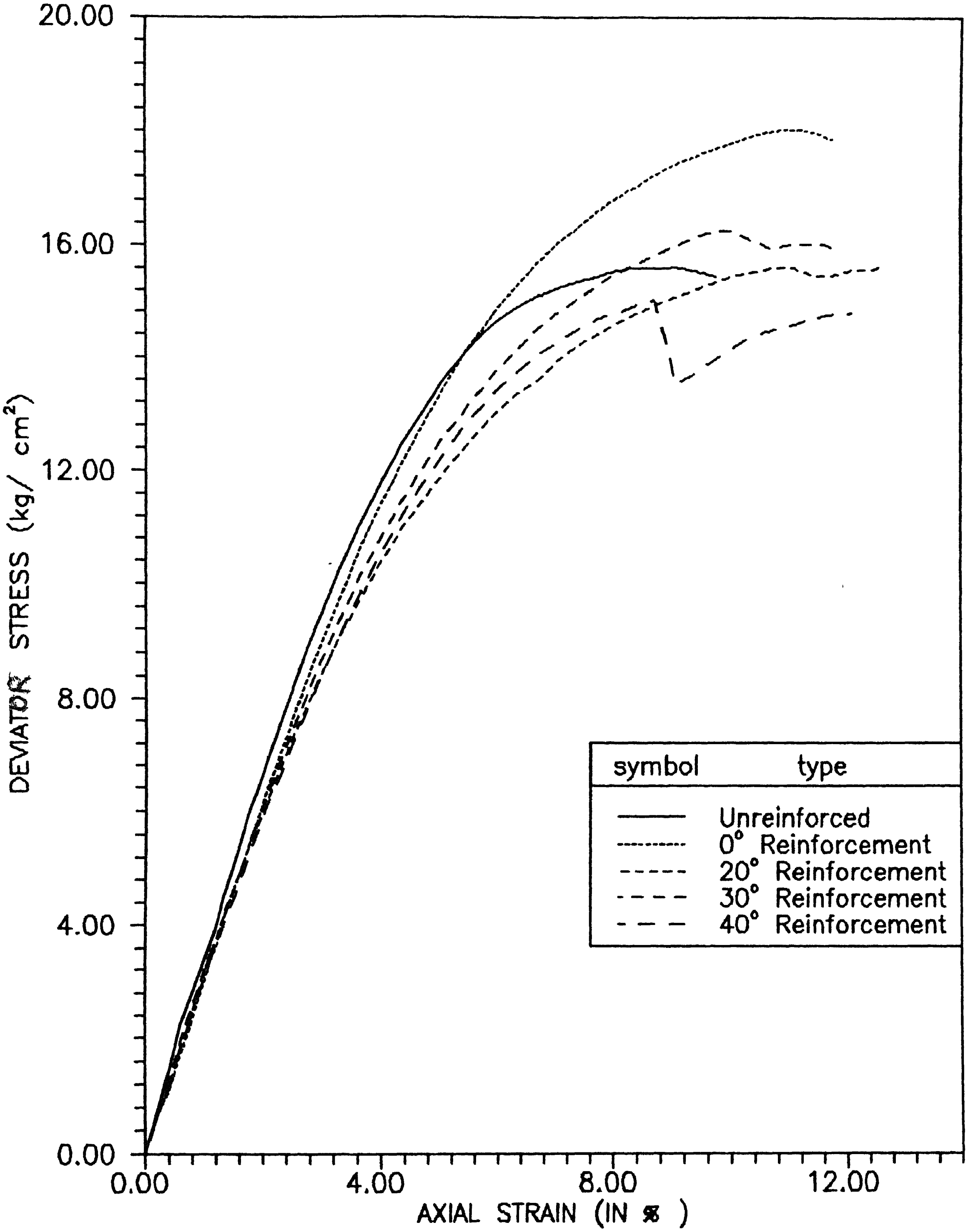


Fig 5.10: Deviator Stress vs Axial Strain Curve of Soil
(For Confining stress = 4.5 kg/cm²)

CHAPTER-6

CONCLUSIONS

Triaxial tests have been conducted with fly ash samples reinforced with polystron geotextiles. The angle of placement of reinforcement have been changed. Following are the conclusions drawn from this study.

1. The maximum improvement of strength in a triaxial test is achieved when the reinforcement is placed in plane of principal tensile strain; which is horizontal in triaxial compression testing.

2. The strength of reinforced soil is maximum when it is placed horizontally in the sample and it decreases with increase in angle to horizontal. At 40° inclination of reinforcement, strength of reinforced soil was found less than strength of unreinforced soil.

3. When reinforcement, which posses only tensile strength, is placed in direction of compressive strain, the strength of ~~strength of~~ reinforced soil will be less than strength of unreinforced soil.

4. The increase of strength is dependent on mobilized tensile resistance in reinforcement and it depends on angle of reinforcement and poisson's ratio of soil. In present study, mobilized tensile resistance at failure was found only in order of 25% of its breaking strength.

5. In this study, failure of reinforced soil is due to compression failure of soil and it is governed by shear strength of soil mass, since reinforcement does not rupture only get stretched.

6. The failure strain was found more at horizontally reinforced soil compare to unreinforced soil and it decreases with increasing the inclination of reinforcement. At 40^{o} inclination of reinforcement, failure strain is less than that of unreinforced soil.

7. The maximum major principal stress value at failure as predicted by using equation 5.10, which includes cohesion term also, is found to be within 15 % of the experimentally measured value.

6.2 RECOMMENDATIONS FOR FUTURE RESEARCH:

1. Experiments with more number of inclined reinforcement layers in soil should be carried out.

2. To analyze the behaviour of reinforced soil, when it fails due to reinforcement rupture or reinforcement slippage.

3. The experimental study should be conducted with other type of reinforcement and soil at different confining stresses.

4. Large scale model test should be carried out on reinforced soil with inclined reinforcement.

REFERENCES

1. Al Refai, T.O.,(1991), "Behaviour of Granular Soils Reinforced with descrete randomly oriented inclusion", *Geotextile and Geomembrane*, Vol.10, No.4, 319-333.

2. Andrawes, K.J., McGown, A., Mashhour, M. and Wilson Faheny, R.F.(1980), "Tension Resistant Inclusions in Soils", *Journal of Geotechnical Engineering Division, ASCE*, Vol.106, No.GT12, 1313-1326.

3. Bengt, B.B., (1988), "Fabric Reinforced Retaining Walls", *Proc. of International Geotechnical Symposium on Theory and Practice of Earth Reinforcement*, Fukvoka, Japan, 3-31.

4. Bishop, A.W. and Henkel, D.J. (1962), "The measurement of soil properties in the triaxial test", *Edward Arnold (Publishers) Limited.*

5. Chandrasekaron,B., Broms, B. B. and Wong, K.S. (1989), "Strength of Fabric Reinforced Sand under axissymmetric loading", *Geotextiles and Geomembranes*, 8, 293-310.

6. Chen, R.H. and Lee, R.G. (1990), "The mechanical effect of inclinations of geotextiles in sands", *An International Conference on Geotextile, Geomembrane and Relatd Products*, Hague, Vol.2, 797.

7. Florkeiwicz, A. 91990), "Bearing Capacity of subsoil with a lauer of reinforced earth", *4th International Conference on Geotextile, Geomembrane and Related Products*, Hague, Vol.1, 162.

8. Fukushima, S., Muchizuki, Y. and Kagawa, K., (1988), *Proc. of International Geotechnical Symposium on Theory and Practice of Earth Reinforcement*, Fukvoka, Japan, 93-98.

9. Gray, D.H.,Kaldjian, M.A. and Wu, C. (1989), "Stress-deformation response of geotextile reinforced", *Geosynthetic Conference Proceedings*, San Diego, Vol.2, 373-384.

10. Gray, D.H., ATHENESOPOULOS G., and Ohashi, H. (1982), "Internal/External fabric reinforcement of sand", *IInd International Conference on Geotextiles*, Log Vegas, 611-616.

11. Haussmann, M.K. (1990), "Engineering Principles of Ground Modifications", *Mcgraw-Hill Publishing Company.*

12. Haussmann, M.K. (1976), "Strength of reinforced soil", *Proceedings Australian road research board*, Vol.8, 1-8.

13. Haussmann, M.K. and Lee, K.L. (1976), "Strength charecteristics of reinforced soil", *International Symposium on New Horizons in Construction materials*, Lehigh University.

14. Hayashi, S., Ochiai, H., Yoshimoto, A., (1988), *Proc. of International Geotechnical Symposium on Theory and Practice of Earth Reinforcement*, Fukvoka, Japan, 99-104.

15. Holtz, R.D., TObin, W.R. and Burke, W.W., (1982), "Creep charecteristics and stress-strain behaviour of a geotextile -reinforced sand", *IInd International Conference on Geotextiles*, Vol.3, Las Vegas, 805-809.

16. Ingold, T.S. and Miller, K.S., (1983), "Drained Axisymmetric

Loading of Reinforced Clay", *Journal Geotechnical Engineering*, 109(7), 883-898.

17. Ingold, T.S. (1979), "Reinforced Clay - A Preliminary Study Using the triaxial apparatus", *International Conference on soil reinfircement*, Vol.1, 59-64.

18. Ingold, T.S., (1982), "An Analytical Study of Geotextile Reinforced Embankment",Las Vegas, Vol.3, 683-688.

19. Jewell, R.A. and Wroth, C.P., (1987), "Direct Shear Tests on Reinforced Sand", *Geotechnique*, Vol.37, 37-44.

20. Koerner, R.M., "Designing with Geosynthetics", *Prentice Hall.*

21. Krishnaswamy, N.R., Reddy, N., (1988), "Behaviour of Reinforced Earth Under Triaxial Compression", *Proc. of the First Indian Geotextiles Conference on Reinforced Soil and Geotextiles*, A41-A46.

22. Krishnaswamy, N.R., Raghavendra, H.B., (1988), "Behaviour of Reinforced Earth under Direct Shear Tests", *Proc. of the First Indian Geotextiles Conference on Reinforced Soil and Geotextiles*, C47-C52.

23. lambe, T.L. and Whitman, R.V. (1969), "Soil Mechanics", *John Wiley and Sons.*

24. Makivichi, K. and Miyamari, T., (1988), *Proc. of International Geotechnical Symposium on Theory and Practice of Earth Reinforcement*, Fukvoka, Japan, 129-134.

25. Mandal, J.N., (1986), *IIIrd International Conference on Geotextiles*, Vienna, 747-750.

26. Mandal, J.N., (1987), "Strength Charecterstics of Soil Reinforcement", *Indian Geotechnical Journal*, 324-334

27. Mandal, J.N., and Divshikar, D.G., (1988), "Large Scale Box Shear Tests with Inclined Reinforcement", *Proc. of the First Indian Geotextiles Conference on Reinforced Soil and Geotextiles*, Vol.1, 173-177.

28. McGown, A.. Andrawes, K.Z. and Al-Hasani,M.M., "Effect of inclusion properties on the behaviour of sand", (1978), *Geotechnique*, 28 (3), 327-346.

29. Mitchell, J.K. and Schlosiser, F. (1979), "General Report", *International Conference of Soil Reinfircement*, Paris, Vol.III, 25-62.

30. Murtaza, G., Shah, S., and Ahamad, M., (1988), "mprovement of Load Bearing Characteristics of Flyash Using Geotextiles", *Proc. of the First Indian Geotextiles Conference on Reinforced Soil and Geotextiles*,F59-F63.

31. Narain, J., (1985), "Reinforced Earth" *Indian Geotechnical Journal*, Vol.15, No.1, 1-25.

32. Rao, G.V. and Raju, G.V.S.S. (1990), "Engineering with Geosynthetics", *Tata Mcgraw-Hill Publishing Company.*

33. Robert, H.S., Rodalph, B.,Robert, C.S., Charles, A.R. and Deniel, R.S., (1991), "Effect of Soil Compaction Conditions on Geomembrane-Soil Interface Strength", *Geotextile and Geomembrene*, Vol.10, No.5.6, 523-529.

34. Schlosser, F. and Long, N.T. (1974), "Recent Results in French

Research on Reinforced Earth" *Journal of Construction Division, ASCE*, Vol.100, No.103, 223-237

35. Schlosser, F., Jacobsen, H.M., and Juran, L., (1983), "Soil Reinforcement", *General Report, Proc. of the VII Euoropean Conference on Soil Mechanics and Foundation Engineering, Helsinki*, Vol.3, 1159-1180.

36. Wu, T.H. (1989), "Behaviour of Geotextiles composits and its application to finite element analysis", *Geosynthetic Conference Proceedings*, San Diego, Vol.2, 365-372.